THE CRITICAL DIFFERENCE SEISMIC DESIGN CAN MAKE FOR A REINFORCED CONCRETE BUILDING LOCATED IN OXFORD, MISSISSIPPI

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# ABSTRACT

The Senior Design course at the University of Mississippi requires the completion of a capstone project. One part of the capstone project is a complete structural design of the NOLA, an ongoing construction in the city of Oxford in Mississippi. The course requires the NOLA to be designed according to gravity loads, meaning the forces act in the downward direction towards the earth's surface. Some loading mechanisms that could occur in a different direction than gravity are seismic loads, which are earthquake-generated loads.

I was not required by my instructors to consider the effect of an earthquake on the structural design of the NOLA, neither was I given enough information to do so. Therefore, I decided to investigate -on my own- how crucial it would be to actually consider the effect of an earthquake on a building in Oxford by researching and consulting the most recent versions of the International Building Code and the American Society of Civil Engineers Minimum Design Standards, I discovered that it is against the guidelines to ignore the effect of an earthquake with the given seismic activity parameters for the city of Oxford. I wanted to investigate and find out the reason behind why it's not permitted to ignore the earthquake effect. Consequently, I would be able to prove that it is vital to consider an earthquake effect in the structural design of any building in Oxford.

I structurally analyzed one frame of the building using two load cases: the first loading case does not consider earthquake activity and the second one does. The frame was then designed to withstand only gravity loads. The results indicated that an earthquake could have a tremendous impact on the design of the structure. An effect significant enough to demolish the initial design with no earthquake activity consideration.

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LIST OF ABBREVIATIONS

R_n – a term used in required percentage of steel expression for vertical members

Φ – safety factor for beam bending design, a value equal to 0.9 for a 10% reduction Using R_n, the steel fraction percentage may be found. Which could be used to select the reinforcing steel bars at the tensile zone of the beam.

ρ – steel fraction percentage of the concrete area

f'_c – specified compressive strength of concrete

l – clear span length

f_y – specified yield strength of non-prestressed reinforcing

A_s – Area of steel needed

b_w – width of the beam

k – effective length factor

l_u – unbraced length of the column

M_1 – smaller moment applied to column

M_2 – bigger moment applied to column

$\emptyset P_n$ – factored nominal axial load

D – Dead load

L – Live load

b – base width of the beam

d – distance from the top of the beam to the centroid of the bottom reinforcement or the distance from the bottom of the beam to the centroid of the top reinforcement

M_u – Ultimate moment experienced by the beam

r – radius of gyration

S_s – mapped MCER, 5% damped, spectral response acceleration parameter at short periods

S_1 – mapped MCE_R, 5% damped, spectral response acceleration parameter at a period of 1 second

S_{DS} – design, 5% damped, spectral response acceleration parameter at short periods

S_{D1} – design, 5% damped, spectral response acceleration parameter at a period of 1 second

R –response modification coefficient for lateral loading

Q_E – effect of horizontal seismic (earthquake-induced) forces

h_x – the height above the base to level x, respectively

C_d – deflection amplification factor

C_u – the coefficient for upper limit on calculated period

C_t – building period coefficient

C_s – Seismic response coefficient

C_{vx} – vertical distribution factor

I_e – The seismic importance factor, a factor that accounts for the degree of risk to human life, health, and welfare associated with damage to property or loss of use or functionality

T_a – approximate fundamental period of the building

δ_{xe} – deflection of Level x at the center of the mass at and above Level x determined by an elastic analysis

δ_x – deflection of Level x at the center of the mass at and above Level x

F_x – portion of the seismic base shear, V, induced at Level i, n, or x,

V – total design lateral force or shear at the base

V_x – seismic design shear in story x

Δ – design story drift

Δ_a – allowable story drift

CHAPTER 1

1.1 Introduction

Reinforced concrete will be used as the building material. Thus, the building will be composed of a slab, beam and column layout. Concrete is typically among the best materials to use for construction since it is unreactive, durable and strong in compression. In the Civil Engineering work force, the first step to begin the design of a building is to acquire and calculate the loading on each frame. Knowing the exact amount of load, with safety and LRFD factors included, the engineer could design each section as economically as possible. The goal behind the design is to create a building that is just strong enough to become structurally stable in the real world.

A group of structural engineers usually work on this type of task, where one group is responsible for determining the loads and another uses those loads to design. This process is rather very time-efficient given the futuristic technology engineers can now utilize to perform three-dimensional modelling of an entire structure.

With the rapidly growing infrastructure, structural engineers who design with concrete always refer to the American Concrete Institute (ACI), American Society of Civil Engineers (ASCE), and the International Building Code (IBC). Those codes provide rules and guidelines as permitted by law to design any structure. This ensures the work ethic of the engineers analyzing and designing the building is up to pace with the first priority being the safety and welfare of the public.

By showcasing a comparison between the most critically affected beam and column design due to an earthquake, there would be solid evidence of the significant change in bending moment, axial force and shear resistance that is needed to completely assure the structural safety of the NOLA in the city of Oxford in Mississippi.

1.2 Project Outline

Using the American Concrete Institute code (ACI), the column and beam placements were chosen accordingly to accommodate for the residential and commercial spaces of the building.

The structure is being designed as a moment-frame system, a special type of frame connection that uses rigid connectors between each of its integral members [1]. With many beams and columns spread around the structure, it is important to understand how much loading transfers to each frame. Since the building is essentially symmetric in dimensions, one frame will be selected for the purpose of analysis:

Figure [1] – Architectural Top View of The NOLA [2]

Figure [2] – Residential Rendering View of The NOLA [2]

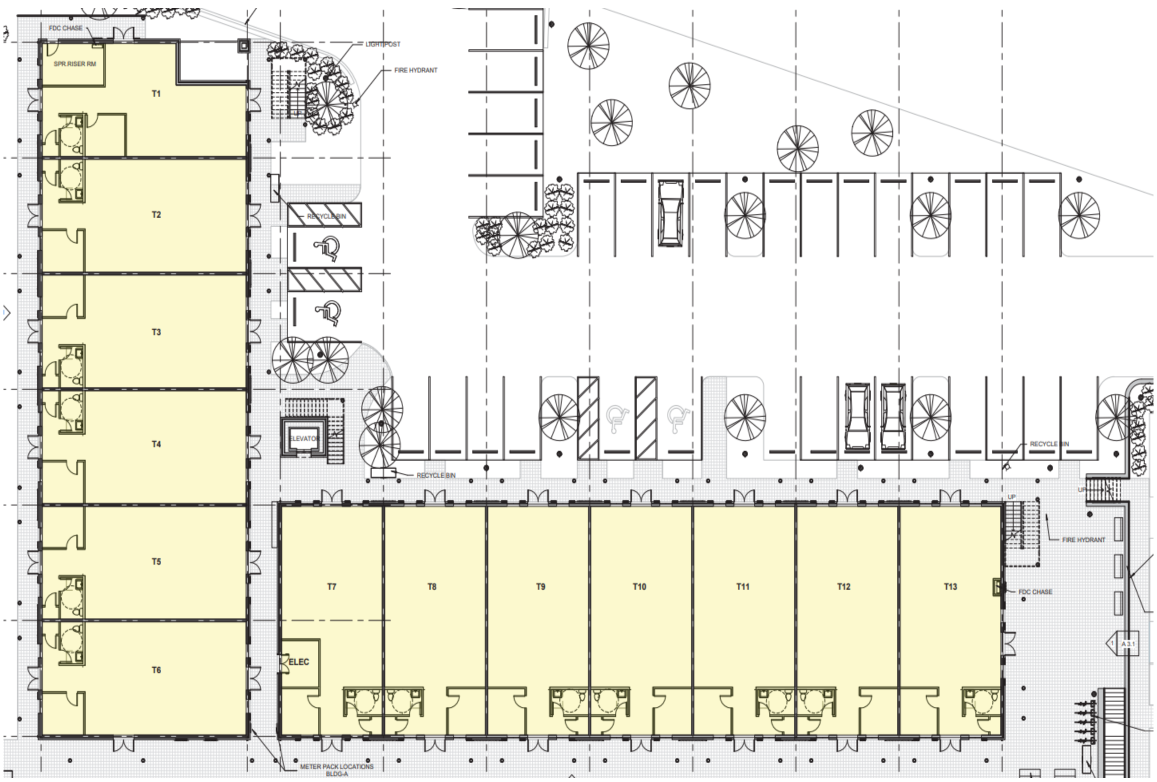

Figure [3] – Architectural Drawing of The NOLA [2]

Figure [4] – Selected Frame for Structural Analysis [2]

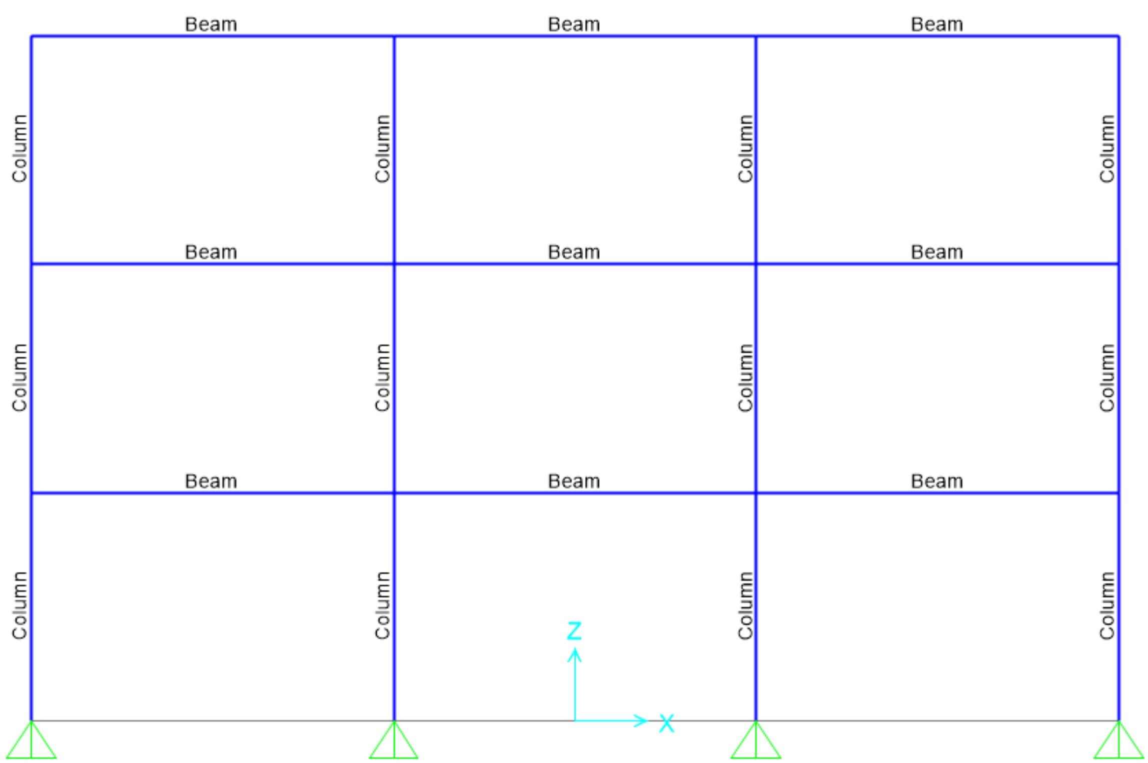

Figure [5] – 2-Dimensional View of the Selected Frame

CHAPTER 2

2. 1 Loading Calculations

2.1.1 Gravity Loads

Dead load will be assumed to be the weight of any material placed on top of the concrete slab and self-weight:

Table 2.1 – Dead Load Calculations [3]

Material	**γ(pcf)**	**Thickness(ft)**	**Area Load (psf)**
Ceramic Tile	138	0.02	2.76
Bedding Mortar	135	0.1	13.5
Lead Conc.	120	0.2	24
Ethafoam	4	0.015	0.06
RC	150	0.7	105
Plaster	140	0.05	7
SUMMATION			152.32

The second and third floors are mainly used for residential purposes. The roof also serves as a place of social setting for the residents to meet so it will be assumed to be for residential purposes as well. The first floor however- is commercial and features mostly retail stores.

Table 2.2 – Minimum Uniformly Distributed Live Loads and Minimum Concentrated live loads [4]

Occupancy or Use	Uniform, L_o psf (kN/m^2)	Live Load Reduction Permitted? (Sec. No.)	Multiple-Story Live Load Reduction Permitted? (Sec. No.)	Concentrated lb (kN)	Also See Section
Penal institutions					
Cell blocks	40 (1.92)	Yes (4.7.2)	Yes (4.7.2)		
Corridors	100 (4.79)	Yes (4.7.2)	Yes (4.7.2)		
Recreational uses					
Bowling alleys, poolrooms, and similar uses	75 (3.59)	No (4.7.5)	No (4.7.5)		
Dance halls and ballrooms	100 (4.79)	No (4.7.5)	No (4.7.5)		
Gymnasiums	100 (4.79)	No (4.7.5)	No (4.7.5)		
Residential					
One- and two-family dwellings					
Uninhabitable attics without storage	10 (0.48)	Yes (4.7.2)	Yes (4.7.2)		4.12.1
Uninhabitable attics with storage	20 (0.96)	Yes (4.7.2)	Yes (4.7.2)		4.12.2
Habitable attics and sleeping areas	30 (1.44)	Yes (4.7.2)	Yes (4.7.2)		
All other areas except stairs	40 (1.92)	Yes (4.7.2)	Yes (4.7.2)		
All other residential occupancies					
Private rooms and corridors serving them	40 (1.92)	Yes (4.7.2)	Yes (4.7.2)		
Public rooms	100 (4.79)	No (4.7.5)	No (4.7.5)		
Corridors serving public rooms	100 (4.79)	Yes (4.7.2)	Yes (4.7.2)		
Stores					
Retail					
First floor	100 (4.79)	Yes (4.7.2)	Yes (4.7.2)	1,000 (4.45)	
Upper floors	75 (3.59)	Yes (4.7.2)	Yes (4.7.2)	1,000 (4.45)	
Wholesale, all floors	125 (6.00)	No (4.7.3)	Yes (4.7.3)	1,000 (4.45)	
Vehicle barriers				See Sec. 4.5.3	
Walkways and elevated platforms (other than exit ways)	60 (2.87)	Yes (4.7.2)	Yes (4.7.2)		
Yards and terraces, pedestrian	100 (4.79)	No (4.7.5)	No (4.7.5)		

Table 2.3 – Final Live Load Values for Selected Frame

Total live load	40 psf

2.1.2 Lateral Loads

According to the scope of the International Building Code [5], there lies no exemption for the structure to be designed to resist the effects of earthquake motion. Therefore, the equivalent lateral force procedure will be used to evaluate the design lateral seismic force.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Seismic

Site Soil Class: C - Very Dense Soil and Soft Rock

Results:

S_S :	0.416	S_{D1} :	0.179
S_1 :	0.179	T_L :	12
F_a :	1.3	PGA :	0.218
F_v :	1.5	PGA_M :	0.261
S_{MS} :	0.54	F_{PGA} :	1.2
S_{M1} :	0.268	I_e :	1.25
S_{DS} :	0.36	C_v :	0.877

Seismic Design Category C

Figure [6] – ASCE Seismic Report Data for Oxford, MS [6]

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Table 2.4 – Risk Category of Buildings and Other Structures [7]

RISK CATEGORY	NATURE OF OCCUPANCY
I	Buildings and other structures that represent a low hazard to human life in the event of failure, including but not limited to: • Agricultural facilities. • Certain temporary facilities. • Minor storage facilities.
II	Buildings and other structures except those listed in Risk Categories I, III and IV
III	Buildings and other structures that represent a substantial hazard to human life in the event of failure, including but not limited to: • Buildings and other structures whose primary occupancy is public assembly with an occupant load greater than 300. • Buildings and other structures containing elementary school, secondary school or day care facilities with an occupa load greater than 250. • Buildings and other structures containing adult education facilities, such as colleges and universities, with an occupant load greater than 500. • Group I-2 occupancies with an occupant load of 50 or more resident care recipients but not having surgery or emergency treatment facilities. • Group I-3 occupancies. • Any other occupancy with an occupant load greater than 5,000[a]. • Power-generating stations, water treatment facilities for potable water, waste water treatment facilities and other public utility facilities not included in Risk Category IV. • Buildings and other structures not included in Risk Category IV containing quantities of toxic or explosive materials that: Exceed maximum allowable quantities per control area as given in Table 307.1(1) or 307.1(2) or per outdoor control area in accordance with the *International Fire Code*; and Are sufficient to pose a threat to the public if released [b].
IV	Buildings and other structures designated as essential facilities, including but not limited to: • Group I-2 occupancies having surgery or emergency treatment facilities. • Fire, rescue, ambulance and police stations and emergency vehicle garages. • Designated earthquake, hurricane or other emergency shelters. • Designated emergency preparedness, communications and operations centers and other facilities required for emergency response. • Power-generating stations and other public utility facilities required as emergency backup facilities for Risk Category IV structures. • Buildings and other structures containing quantities of highly toxic materials that: Exceed maximum allowable quantities per control area as given in Table 307.1(2) or per outdoor control area in accordance with the *International Fire Code*; and Are sufficient to pose a threat to the public if released [b]. • Aviation control towers, air traffic control centers and emergency aircraft hangars. • Buildings and other structures having critical national defense functions. • Water storage facilities and pump structures required to maintain water pressure for fire suppression.

Table 2.5 – Design Coefficients and Factors for Seismic Force-Resisting Systems [8]

Seismic Force-Resisting System	ASCE 7 Section Where Detailing Requirements Are Specified	Response Modification Coefficient, R^a	Overstrength Factor, Ω_0^b	Deflection Amplification Factor, C_d^c	Structural System Limitations Including Structural Height, h_n (ft) Limits[d] — Seismic Design Category: B	C	D^e	E^e	F^f
A. BEARING WALL SYSTEMS									
1. Special reinforced concrete shear walls[g,h]	14.2	5	2½	5	NL	NL	160	160	100
2. Ordinary reinforced concrete shear walls[g]	14.2	4	2½	4	NL	NL	NP	NP	NP
3. Detailed plain concrete shear walls[g]	14.2	2	2½	2	NL	NP	NP	NP	NP
4. Ordinary plain concrete shear walls[g]	14.2	1½	2½	1½	NL	NP	NP	NP	NP
5. Intermediate precast shear walls[g]	14.2	4	2½	4	NL	NL	40[i]	40[i]	40[i]
6. Ordinary precast shear walls[g]	14.2	3	2½	3	NL	NP	NP	NP	NP
7. Special reinforced masonry shear walls	14.4	5	2½	3½	NL	NL	160	160	100
8. Intermediate reinforced masonry shear walls	14.4	3½	2½	2¼	NL	NL	NP	NP	NP
9. Ordinary reinforced masonry shear walls	14.4	2	2½	1¾	NL	160	NP	NP	NP
10. Detailed plain masonry shear walls	14.4	2	2½	1¾	NL	NP	NP	NP	NP
11. Ordinary plain masonry shear walls	14.4	1½	2½	1¼	NL	NP	NP	NP	NP
12. Prestressed masonry shear walls	14.4	1½	2½	1¾	NL	NP	NP	NP	NP
13. Ordinary reinforced AAC masonry shear walls	14.4	2	2½	2	NL	35	NP	NP	NP
14. Ordinary plain AAC masonry shear walls	14.4	1½	2½	1½	NL	NP	NP	NP	NP
15. Light-frame (wood) walls sheathed with wood structural panels rated for shear resistance	14.5	6½	3	4	NL	NL	65	65	65
16. Light-frame (cold-formed steel) walls sheathed with wood structural panels rated for shear resistance or steel sheets	14.1	6½	3	4	NL	NL	65	65	65
17. Light-frame walls with shear panels of all other materials	14.1 and 14.5	2	2½	2	NL	NL	35	NP	NP
18. Light-frame (cold-formed steel) wall systems using flat strap bracing	14.1	4	2	3½	NL	NL	65	65	65
B. BUILDING FRAME SYSTEMS									
1. Steel eccentrically braced frames	14.1	8	2	4	NL	NL	160	160	100
2. Steel special concentrically braced frames	14.1	6	2	5	NL	NL	160	160	100
3. Steel ordinary concentrically braced frames	14.1	3¼	2	3¼	NL	NL	35[j]	35[j]	NP[j]
4. Special reinforced concrete shear walls[g,h]	14.2	6	2½	5	NL	NL	160	160	100
18. Ordinary reinforced masonry shear walls	14.4	2	2½	2	NL	160	NP	NP	NP
19. Detailed plain masonry shear walls	14.4	2	2½	2	NL	NP	NP	NP	NP
20. Ordinary plain masonry shear walls	14.4	1½	2½	1¼	NL	NP	NP	NP	NP
21. Prestressed masonry shear walls	14.4	1½	2½	1¾	NL	NP	NP	NP	NP
22. Light-frame (wood) walls sheathed with wood structural panels rated for shear resistance	14.5	7	2½	4½	NL	NL	65	65	65
23. Light-frame (cold-formed steel) walls sheathed with wood structural panels rated for shear resistance or steel sheets	14.1	7	2½	4½	NL	NL	65	65	65
24. Light-frame walls with shear panels of all other materials	14.1 and 14.5	2½	2½	2½	NL	NL	35	NP	NP
25. Steel buckling-restrained braced frames	14.1	8	2½	5	NL	NL	160	160	100
26. Steel special plate shear walls	14.1	7	2	6	NL	NL	160	160	100
C. MOMENT-RESISTING FRAME SYSTEMS									
1. Steel special moment frames	14.1 and 12.2.5.5	8	3	5½	NL	NL	NL	NL	NL
2. Steel special truss moment frames	14.1	7	3	5½	NL	NL	160	100	NP
3. Steel intermediate moment frames	12.2.5.7 and 14.1	4½	3	4	NL	NL	35[k]	NP[k]	NP[k]
4. Steel ordinary moment frames	12.2.5.6 and 14.1	3½	3	3	NL	NL	NP[l]	NP[l]	NP[l]
5. Special reinforced concrete moment frames[m]	12.2.5.5 and 14.2	8	3	5½	NL	NL	NL	NL	NL
6. Intermediate reinforced concrete moment frames	14.2	5	3	4½	NL	NL	NP	NP	NP
7. Ordinary reinforced concrete moment frames	14.2	3	3	2½	NL	NP	NP	NP	NP
8. Steel and concrete composite special moment frames	12.2.5.5 and 14.3	8	3	5½	NL	NL	NL	NL	NL
9. Steel and concrete composite intermediate moment frames	14.3	5	3	4½	NL	NL	NP	NP	NP
10. Steel and concrete composite partially restrained moment frames	14.3	6	3	5½	160	160	100	NP	NP
11. Steel and concrete composite ordinary moment frames	14.3	3	3	2½	NL	NP	NP	NP	NP
12. Cold-formed steel—special bolted moment frame[n]	14.1	3½	3[o]	3½	35	35	35	35	35

Table 2.6 – Importance Factors by risk Category of Buildings and Other Structures for Snow, Ice, and Earthquake Loads [9]

Risk Category from Table 1.5-1	Snow Importance Factor, I_s	Ice Importance Factor—Thickness, I_i	Ice Importance Factor—Wind, I_w	Seismic Importance Factor, I_e
I	0.80	0.80	1.00	1.00
II	1.00	1.00	1.00	1.00
III	1.10	1.15	1.00	1.25
IV	1.20	1.25	1.00	1.50

$$S_{DS} = \frac{2}{3} S_{MS} \quad \text{[Eqn. 1]} \qquad [10]$$

$$S_{DS} = \frac{2}{3} \times 0.54 = 0.36$$

$$S_{D1} = \frac{2}{3} S_{M1} \quad \text{[Eqn. 2]} \qquad [11]$$

$$S_{DS} = \frac{2}{3} \times 0.268 = 0.18$$

Table 2.7 – IBC Risk Category Determination

IBC Specification	Risk Category
0.33 g ≤ S_{DS} < 0.5 g	C
0.133 g ≤ SD1 < 0.20 g	C
Most Severe category	C

$$C_s = \frac{S_{DS}}{(R/I_e)} \quad \text{[Eqn. 3]} \qquad [12]$$

Risk Category is III from Table 2.4

$I_e = 1.25$ from Table 2.6

$R = 3$ from Table 2.5

$C_d = 2.5$ from Table 2.5

Risk Category is C from Table 2.7

$$C_s = \frac{0.18}{(3/1.25)}$$

$$C_s = 0.15$$

$$T_a = C_t h_n^x \text{ [Eqn. 4]} \qquad [13]$$

The effective seismic weight of each floor was calculated using the known dimensions of slabs and the design dimension of the beam and columns from gravity load design.

$$W = 3476.9\ kips$$

$$V = C_s W \text{ [Eqn. 5]} \quad [14]$$

$$V = 0.15 \times 3476.9\ kips$$

$$V = 522.3\ kips$$

The distribution exponent is equal to 1.0 for buildings with an elastic fundamental period less than or equal to 0.5 seconds [15]

The vertical distribution factor must be calculated, which is equal to a percentage of the base shear that is assigned at each floor level.

$$C_{vx} = \frac{w_x h_x^k}{\sum w_i h_i} \text{ [Eqn. 6]} \qquad [16]$$

The previous values computed for lateral load undergo a complicated set of calculations and summations, therefore the best method to continue the calculation procedure is through tabulation of the values:

Table 2.8 – Evaluation of the Seismic Design Shear for Each Story

Story	**W_x (kips)**	**h_x (ft)**	**$W_x h_x^k$**	**C_{vx}**	**F_x (kips)**	**V_x / story (kips)**
1		0.0			0	522
2	522	10.5	5484	0.17	87	435
3	522	21.0	10968	0.33	174	261
Roof	522	31.5	16453	0.50	261	0
SUMMATION	1567		32905		522	

2.2 Slab Thickness

The selected frame from Figure [4] has to carry some of the weight of the slab that is placed on top of it. The weight of slab cannot be determined without knowing how thick the slab needs to be.

Table 2.9 – Minimum Thickness of Non-prestressed Beams or One-Way Slabs Unless Deflections Are Computed [17]

	Minimum thickness, ***h***			
	Simply supported	One end continuous	Both ends continuous	Cantilever
Member	Members not supporting or attached to partitions or other construction likely to be damaged by large deflections			
Solid one-way slabs	**ℓ/20**	**ℓ/24**	**ℓ/28**	**ℓ/10**
Beams or ribbed one-way slabs	**ℓ/16**	**ℓ/18.5**	**ℓ/21**	**ℓ/8**

The slab placed on top of the frame is a solid one-way slab since the length to width ratio is greater than 2.0. From Table 2.9, the one end continuous equation governs the minimum thickness permissible. Therefore:

$$h = \frac{l}{24} \text{ [Eqn. 7]}$$

$$\mathrm{h} = \frac{(16.67\ \mathrm{ft\ x}\ \frac{12\ in}{1\ ft})}{24} = 8.335\ in, use\ 8.5\ in.$$

2.3 Frame Loading Calculations

2.3.1 Tributary Area

The selected frame from Figure 5 partially carries part of the area loading. The amount of load that the frame carries will be calculated using the tributary area method. Since the beams are placed in 4 directions, the short beams carry a triangular area with 45 degree sides and the remainder of the area is carried by the long beams as shown below.

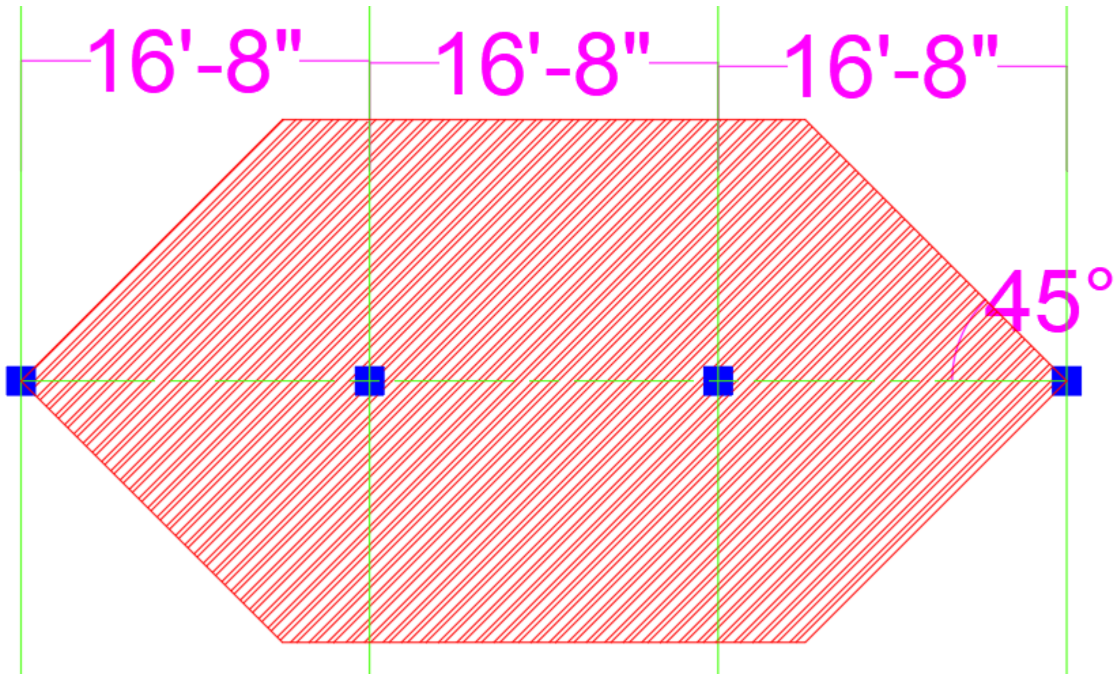

Figure [7] – Tributary Area Technique Applied to the Selected Frame

2.3.2 Dead loading from weight applied to the slab

The area load will be multiplied by half the transverse length between the beams to convert into a linear load as mentioned previously.

$$Dead\ load = 153\frac{lb}{ft^2} \times 933.2\ ft^2\ = 142{,}780\ lb$$

$$Divide\ by\ length\ of\ beam = \frac{142{,}780\ lb}{50\ ft} = 2{,}855.6\ lb$$

2.3.3 Dead loading from slab weight

Based off of Figure 7, the area loading for the dead and live load results can now be converted into a linear load onto the selected frame.

$$Dead\ load = \ 150\ ft\ \times 25\ ft\ \times \frac{8.5}{12}\ ft\ \times \frac{0.15\ k}{ft^3} = 398.44\ kips$$

1 frame carries the total weight of the slab divided by the number of frames in the span subtracted from 1. This is because the edge frames only carry half the weight:

$$\frac{398.44\ k}{7-1} = 66.41\ kips\ distrubuted\ over\ 50\ ft\ frame$$

$$\frac{66.41\ k}{50\ ft} = 1.33\frac{k}{ft} = 1{,}330\frac{lb}{ft}$$

2.3.4 Total Linear Dead Load

$$Total\ Dead\ Load = 2855.6\frac{lb}{ft} + 1{,}330\frac{lb}{ft} = 4185.6\frac{lb}{ft} = 4.19\frac{k}{ft}$$

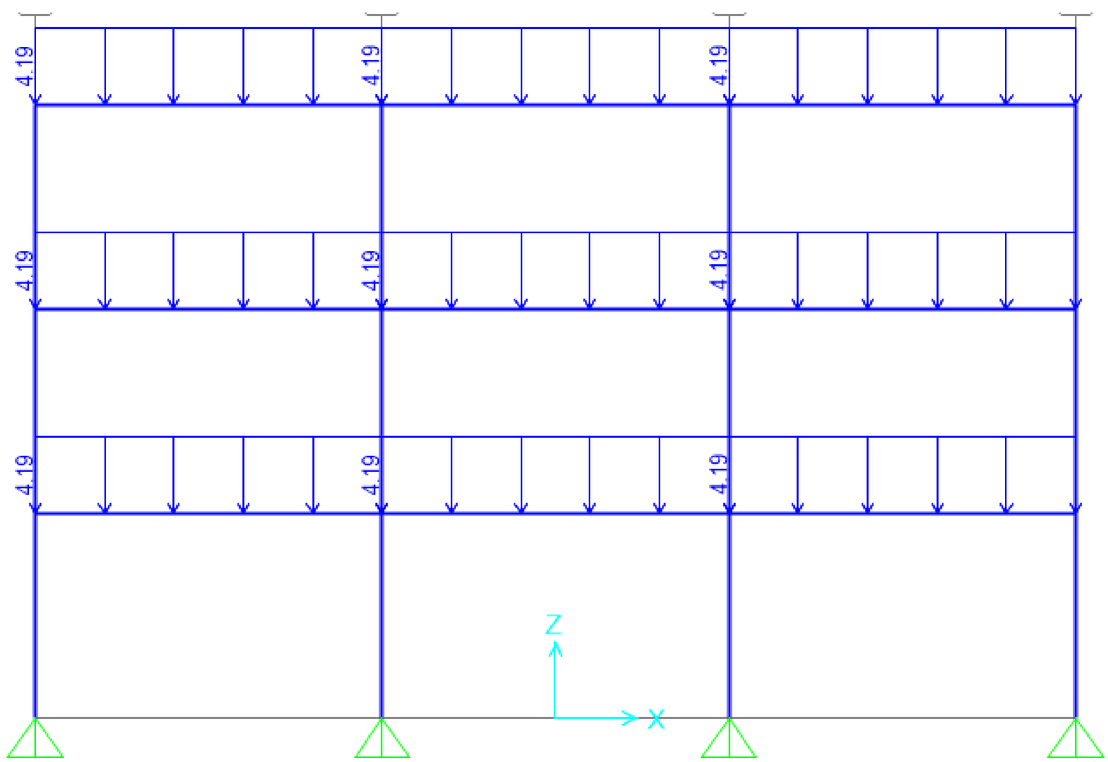

Figure [8] – Total Dead Load Applied to the Selected 2-D Frame

2.3.5 Total Linear Live Load on each floor

From Table 2.3, live loading is 40 lb/ft^2

$$total\ live\ load\ on\ each\ floor\ of\ the\ frame = 40\frac{lb}{ft^2} \times 933.2\ ft^2 = 37{,}328\ lb$$

$$Divide\ by\ length\ of\ frame = \frac{37{,}328\ lb}{50\ ft} = 746.56\ lb = 0.75\ k$$

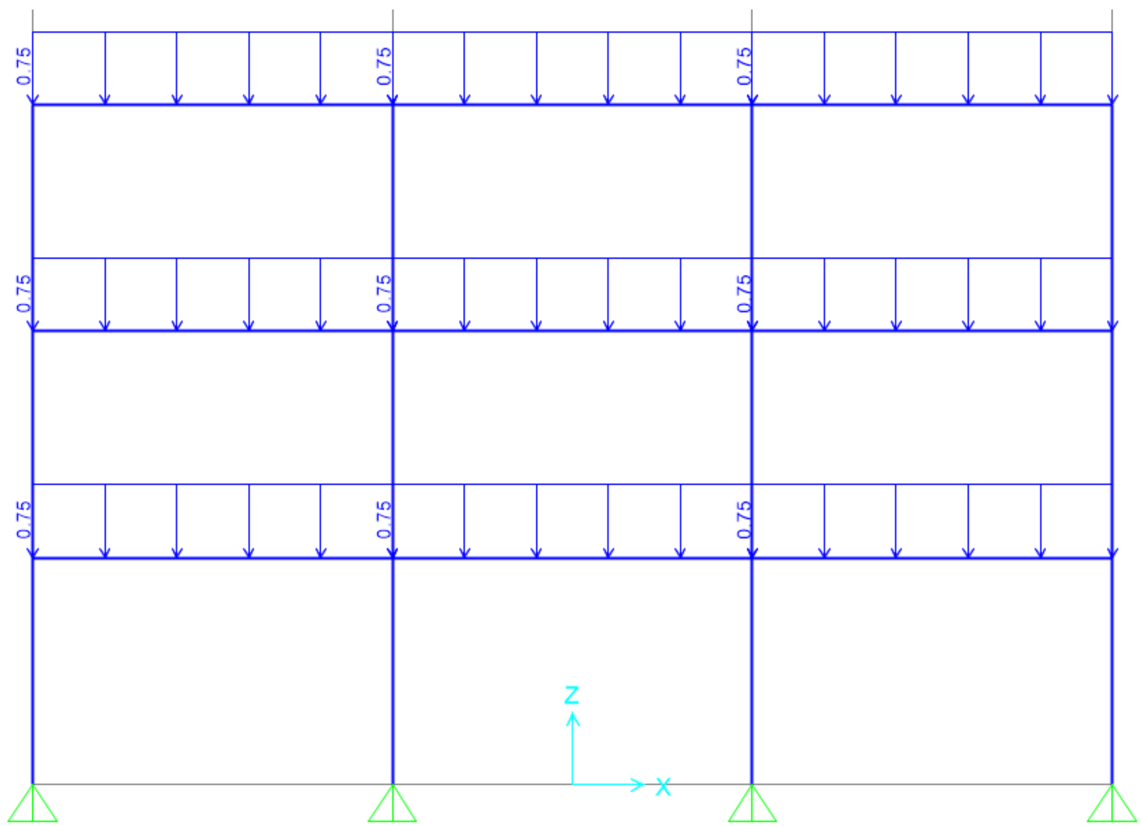

Figure [9] – Total Live Load Applied to the Selected 2-D Frame

2.3.6 Total Joint Lateral Load from seismic effect

After redistribution of the seismic shear across the height of the building as seen in Table 2.8, it can be seen that in a ascending order of stories, the lateral force increases which is logically valid as to which the groundular movement occurs at the fault lines of the earth's surface. Thus, this allows the advancement into the next designing procedure of a ordinary reinforced concrete moment frame.

The lateral force is applied onto the centroid of each beam-slab joint over a two-dimensional analysis on each story, as it was derived from the weight of the entire floor and story height. For structural design purposes, the loads are applied in a distributed way to the entire story, hence avoiding any numerical singularity and overdesigning, as it would be the case if point loads would be applied in a conventional way.

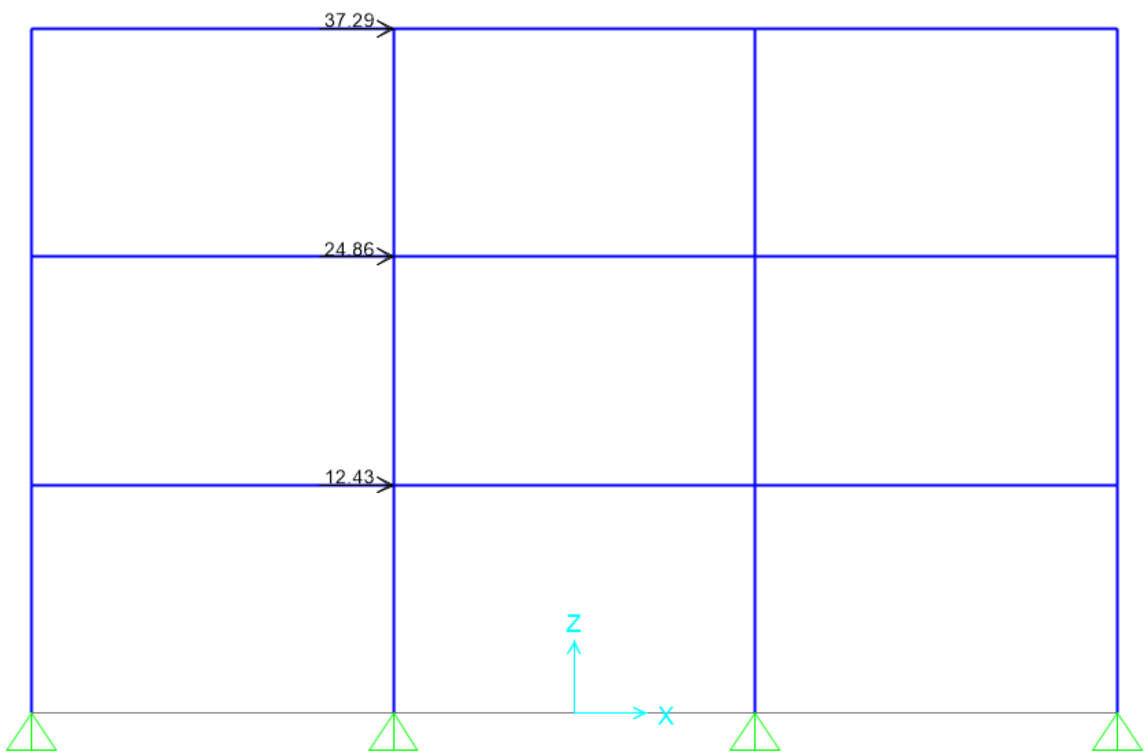

Figure [10] – Distributed Seismic Shear Applied to the Centroid of each Beam of the Selected 2-D frame

CHAPTER 3

3.1 Building Code Requirements

3.1.1 Bending Moment Design in Beams

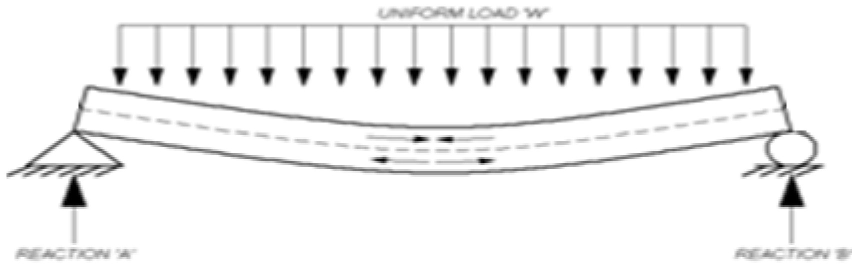

Figure [11] – Bending Beam Example

The structural design of a beam is typically controlled by a bending failure as shown in Figure 11. Upon immense loading onto the span, there will be a moment reaction at the cross section to statically balance the force and prevent the beam from falling to the ground. Upon this bending the reaction, it causes the beam to enter a "smiling" or "frowning" orientation. For the purpose of explanation, assume the loading causes the beam to be in a "smiling" orientation as shown in Figure 11. During that instance of time of a "smiling" orientation, the horizontal length of the beam at the top of the centroidal axis becomes shorter in length than the horizontal length of the beam at the bottom of centroidal axis. This creates a compression at the top as the stress elements are getting closer together, while the stress elements at the bottom of the beam that is increasing in length are actually getting separated and feel a tensile force trying to stop them from separating from each other in the normal equilibrium state. Since concrete is strong in compression but weak in tension, the steel reinforcement bars are strictly placed in the tensile zone, as it simultaneously strengthens the design and is more economic.

Therefore, the first controlling parameter of the beam is a bending due to a moment [19].

$$R_N = \frac{M_u}{\emptyset b d^2} \qquad \text{[Eqn. 8]}$$

$$\rho = \frac{0.85 f'_c}{f_y}\left(1 - \sqrt{1 - \frac{2R_n}{0.85 f'_c}}\right) \qquad \text{[Eqn. 9]}$$

$$A_s = \rho b d \qquad \text{[Eqn. 10]}$$

3.1.2 Shear Design in Beams

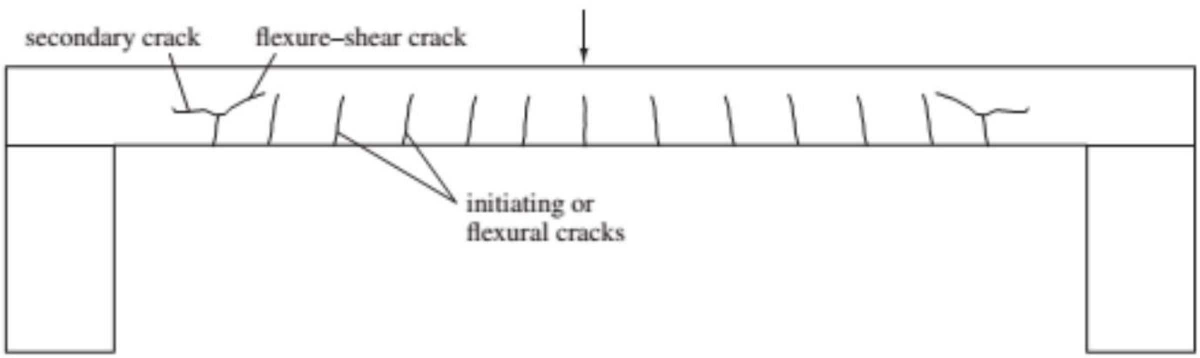

FIGURE 8.1 Flexure–shear crack.

Figure [12] - Demonstration of a Shear Failure in a Beam

Beams could also fail in shear. This is a vertical force parallel to the cross section of the beam which occurs to reach equilibrium after a vertical force application. A stirrup reinforcement is typically used to strengthen the shear strength of the beam [20]. Thus, the final design of the beam will be a combination of stirrups and reinforcing bars. A typical example of the final design cross section of a beam is shown in Figure 13.

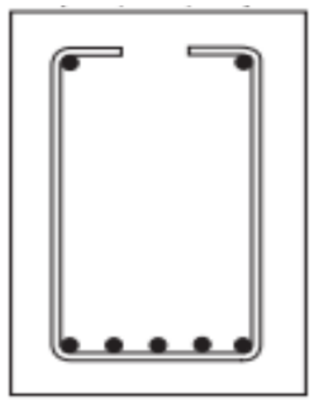

Figure [13] - Example of the Final Design Cross Section of a Beam [21]

The concrete by itself without any stirrups could carry an amount of shear V_c

$$\emptyset V_c = \emptyset 2\lambda\sqrt{f'_c}\, b_w d \quad \text{[Eqn. 11]}$$

λ= modification factor reflecting the reduced mechanic properties of lightweight concrete; all relative to normal weight concrete of the same compressive strength

d = distance from the top of the beam to the centroid of the bottom reinforcement or the distance
from the bottom of the beam to the centroid of the top reinforcement
Φ = safety factor for beam shear design, a value equal to 0.75 for a 25% reduction

ACI code 9.6.3.1 states that stirrups are needed in any region of the beam where V_u may greater than half the factored shear strength capacity of the concrete V_c

$$\frac{1}{2}\emptyset V_c < V_u \qquad \text{[Eqn. 12]}$$

If Equation 12 is satisfied, then stirrups must be added to support the remaining portion of ultimate shear which the concrete by itself cannot handle.

$$\emptyset V_c + \emptyset V_s = V_u \qquad \text{[Eqn. 13]}$$

After solving for V_s, the spacing between the stirrups may be computed using the least of the following parameters given by the ACI code 9.7.6.2.2

$$s = \frac{A_v f_{yt} d}{V_s} \qquad \text{[Eqn. 14]}$$

$$s = \frac{A_v f_{yt}}{0.75\sqrt{f'_c} b_w} \quad \text{[Eqn. 15]}$$

$$s = \frac{A_v f_{yt}}{50 b_w} \quad \text{[Eqn. 16]}$$

$$s = \frac{d}{2} \quad \text{[Eqn. 17]}$$

3.1.3 Beam Column Design

In a structural project, it is important to understand the load path through the members. In general, the slab transfers the load to the beams, which transfer the load to the nearby column. Those stresses travel in the vertical direction until the next beam underneath. This process occurs again for each beam to column until the loading reaches the footing. Regardless of how many floors are present in a building, each column operates on a continuous column, consequently, this means each beam at each floor gives the same reaction at the joint of the continuous column. This is because each floor is made up of the same slab and presumably the same amount of people, thus the slabs receive the same amount of factored load while the continuous column bears those loads individually at each slab joint. Knowingly, this means the columns placed at the floor will have to bear the weight of the whatever part of the structure is above [22].

Before initiating the design process, the slenderness of the column itself must be determined. This is based on the geometry of the column and whether it is laterally braced. Reinforced concrete columns usually have small slenderness ratios. As a result, they can usually be designed as short columns without strength reductions because of slenderness. However, no design shall begin unless the column is proven to be *short* using the slenderness ratio according to the ACI Code requirements [23].

For an unbraced column that is fixed at both ends:

$$k = 0.5 \qquad \text{[Eqn. 18]}$$

For sway frames, the column is considered short if the following equation is satisfied

$$\frac{kl_u}{r} \leq 22 \qquad \text{[Eqn. 19]}$$

(Where that M_1 is the smaller of the two moments. Note that M_1 should have a negative sign for single curvature and a positive sign for double curvature)

(ACI Code section 2.5.6.1 permits to equalize r to 0.3 times the dimension of the rectangular column stability is being considered in)

After determination of each column of the building being short using the Equation 19, the project has been designed using short reinforced concrete columns. Since short columns failure occurs due to initial material failure, the load that it can support is controlled by the dimensions of the cross sections and the strength of the material. This fact has been taken in account and the following equation is used to calculate the gross area of square column.

$$\emptyset P_n = \emptyset 0.80[0.85f'_c(A_g - A_{st}) + f_y A_{st}] \qquad \text{[Eqn. 20]}$$

A_g is no more than the preliminary design area for the column experiencing pure axial loading. In reality however-the columns also experience a moment. An interaction diagram which has been already developed by the ACI code has different design parameters for the steel fractional area based on multiple iterations of assumed steel strain. Thus, the diagram may be used to design a column that undergoes both axial load and bending [24].

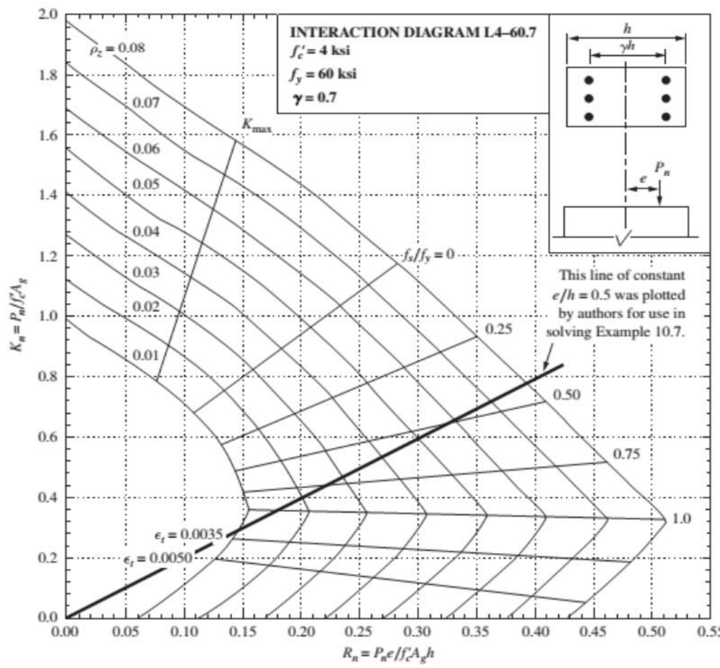

Figure [14] – Example of an Interaction Diagram

3.1.4 Overturning and Deflection

The structure shall be designed to resist overturning effects caused by the seismic forces. Rigid diaphragms are applied to each joint of each story. Rigid diaphragms are structural elements that transmit the lateral loads to the frame.

$$\delta_x = \frac{C_d \delta_{xe}}{I_e} \quad \text{[Eqn. 21]} \quad [25]$$

The drift at story level x is determined by subtracting the design earthquake displacement at the bottom of the story from the design earthquake displacement at the top of the story.

$$\Delta = \delta_x - \delta_{x-1} \quad \text{[Eqn. 22]} \quad [26]$$

The design story drift (Δ) as determined Equation 22 shall not exceed the allowable story drift (Δ_a) as obtained from Table 2.10

Table 2.10 – Allowable Story Drift Δ_a [27]

Structure	Risk Category I or II	Risk Category III	Risk Category IV
Structures, other than masonry shear wall structures, four stories or less above the base as defined in Section 11.2, with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts	$0.025h_{sx}$[c]	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall structures[d]	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
All other structures	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$

$$\Delta_a = 0.020h_{xx} \text{ [Eqn. 23]}$$

If the deflection of any joint of the frame surpasses Equation 23, then the design is not safe. Results for the deflections can be found in Table 4.4.

3.2 Structural Analysis Method

The structural analysis is performed using Load & Resistance Factor Design, commonly referred to as the *LRFD Load Factors* in the structural engineering field. LRFD Load Factors compute for a percentage increase/decrease for multiple combined loading cases. This creates a limit state condition for which if bypassed, the structure no longer fulfills the safety design criteria. Its a method of proportioning structural members such that the computed forces produced in the members by the factored loads do not exceed the member design strength.

Since only dead, live and earthquake loading will be used, the following LRFD [28] equations govern:

$$1.4D \text{ [Eqn. 24]}$$

$$1.2D + 1.6L \text{ [Eqn. 25]}$$

$$1.2\,D + 0.5L + 1.0E \text{ [Eqn. 26]}$$

$$0.9D + 1.0E \text{ [Eqn. 27]}$$

E = Earthquake effect

$$E = \rho QE - 0.2S_{DS}D \quad \text{[Eqn. 28]}$$

Substitute S_{DS} = 0.36

D = Dead load

Substitute ρ = reliability factor = 1 for SCS A, B or C

Through mathematical trial, Equation 26 controls Equation 27. Therefore, by substituting the S_{DS} and D into E, then Substituting E into Equation 27:

$$1.27D + 0.5L + Q_E \quad \text{[Eqn. 29]}$$

CHAPTER 4

4.1 Frame Analysis and Design

It is completely possible to analyze the frame in Figure 5 without the use of technology but it is rather very time consuming. For the purposes of merely accomplishing the objective of this study comparison, a structural analysis software called "SAP2000" will be used to determine the axial force, bending moment, and shear in the critical beam and column of the frame [29]. Within the software itself, the loading cases of Equation 25 & 29 were implemented to demonstrate the results for each case separately.

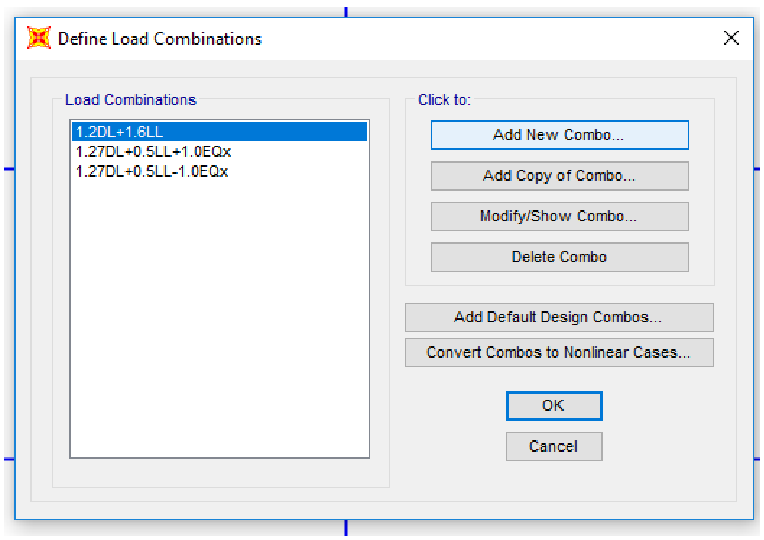

Figure [15] – Load Case Implementation in SAP2000

The frame will be designed based on two sets of results. The first set involves only gravity loads. The second set involves both the gravity loads and lateral loads previously discussed. All the results for each category may be found in the Appendix.

4.1.1 Bending Moment Design

The main goal behind the design of a beam is finding the depth, width and the steel percentage. This can only be done by starting with a reasonable assumption. A good approach is to attempt to minimize the steel area and also to reach an aspect ratio between 1.5-2.0 to maintain strong rigidity. Therefore, the analysis procedure was performed with repetitive trials of assumed frame sections. The depth and width would be decreased by 1 inch in each structural analysis trial until the value of the bending moment began to exceed 240 kip-ft. Thus, the final beam dimensions where chosen to be a 24.5" depth and a 14" base thickness. The cover for the reinforcing bars is 2.5" which is the generic value used in reality.

When designing for the positive moment, the tension occurs at the bottom. Therefore, the maximum moment is used to design for the steel reinforcing bars placed at the bottom of the beam.

From Equation 8, $R_N = \frac{M_u}{\emptyset b d^2}$ and Appendix B3

$$R_N = \frac{12\frac{in}{ft} \times 115{,}500 - ft}{0.9 \times 14\, in \times (24.5\, in)^2} = 182.46\, psi$$

From Equation 9,

$$\rho = \frac{0.85 f'_c}{f_y}(1 - \sqrt{1 - \frac{2R_n}{0.85 f'_c}})$$

$$\rho = \frac{0.85 \times 4000\, psi}{60{,}000\, psi}\left(1 - \sqrt{1 - \frac{2 \times 182.46\, psi}{0.85 \times 4000\, psi}}\right) = 0.00313 < \rho_{min} = 0.0033$$

From Equation 10, $A_s = 0.0033 \times 14\,in \times 24.5\,in = 1.132\,in^2$

Therefore, use 6 #4 bars at the bottom of the beam as they have an area of

1.18 in^2 > 1.132 in^2 [32]

The same calculation was repeated for the critical negative moment of 158.6 k-ft which yields a final A_s of 1.5 in^2. Therefore, use 5 #5 bars at top of the beam as they have an area of 1.53 in^2 > 1.5 in^2 [33]

Final design of reinforcing bars:

6 #4 at the bottom of the beam, 2.5” cover

5 #5 bars at the top of the beam, 2.5” cover

4.1.3 Shear Design

From Equation 11, $\emptyset V_c = \emptyset 2\lambda\sqrt{f'}_c\, b_w d$

$$\emptyset V_c = 0.75 \times 2 \times 1.0 \times \sqrt{4000}\,psi \times 14\,in \times 24.5\,in$$
$$\emptyset V_c = 32539.84\,lb = 32.5\,k$$

This what the concrete by itself can carry without any stirrups, referred to as V_c

As a second level of safety check, the ACI code 9.6.3.1 states that the ultimate shear applied to the beam V_u may not be less than the half the factored shear strength capacity of the concrete V_c

From Equation 12 and Appendices B4-B7:

$$stirrups\ are\ needed\ if\ \frac{1}{2}\emptyset V_c < V_u$$

$$\frac{1}{2}\emptyset V_c = 0.5 \times 32.5\,k = 16.25\,k$$

$$V_u = 45.7\,k\ (from\ Appendix\ B5)$$

$$\frac{1}{2}\emptyset V_c < V_u$$

Therefore, Equation 12 is not satisfied, and stirrups must be added to support the remaining portion of the ultimate shear which the concrete itself cannot handle. From Equation 13:

$$\emptyset V_c + \emptyset V_s = V_u$$

$$\emptyset V_s = V_u - \emptyset V_c$$

$$\emptyset V_s = 45.7 - 32.5\,k = \ 13.2\,k = 13{,}200\,lb$$

$$V_s = \frac{13{,}200\,lb}{\emptyset} = \frac{13{,}200\,lb}{.65} = 20{,}308 = 20.31k$$

The spacing between the stirrups may be computed using the least of the following parameters given by the ACI code 9.7.6.2.211

$$s = \frac{A_v f_{yt} d}{V_s} \qquad \text{[Eqn. 14]}$$

$$s = \frac{A_v f_{yt}}{0.75\sqrt{f'_c}\,b_w} \qquad \text{[Eqn. 15]}$$

$$s = \frac{A_v f_{yt}}{50 b_w} \qquad \text{[Eqn. 16]}$$

$$s = \frac{d}{2}\ or\ 12" \qquad \text{[Eqn. 17]}$$

Where A_v is equal to 0.22 in^2, the area of a #3 stirrup.

After mathematical trials, the least spacing is obtained using Equation 17:

$$s = \ 12"$$

Since the shear value can only get smaller, the spacing can only get bigger. With 12 inches being the max spacing allowed by ACI code, 12 inch spaced #3 stirrups will be used across the beam until termination at the location where the shear experienced is less than $\frac{1}{2}\emptyset V_c$. The stirrup termination location can be found in Appendices B6-B7

Final design of stirrup:

1 #3 at 2”, 4 #3 at 12” spacing starting from left supporting

1 #3 at 2”, 6 #3 at 12” from starting from right support

4.1.5 Column Axial and Bending Moment Design

During the entire structural analysis, 9 inch by 9 inch columns were used. This was no more than an assumption, as the smallest column that could be used is a 9 inch by 9 inch and the building is designed merely as a residential building, thus it mainly just carries occupant weight and self-weight.

For an unbraced column with both ends fixed, Equation 18 states:

$$k = 0.5$$

Using Equation 19, if the inequality is satisfied then the column is confirmed to be short:

$$\frac{kl_u}{r} \leq 34 - 12\frac{M_1}{M_2}$$

$$\frac{0.5 \times (10.5\,ft\ \times 12\frac{in}{ft} - (13.5\,in + 13.5\,in)}{(0.3\ \times\ 9\,in)} \leq 34 - 12(\frac{-11.6\,k - ft}{23.6\,k - ft})$$

$13.333 \leq 22$, column is short

Using Equation 20, Appendices B8-B10, and assuming 30% reinforcement is needed:

$$\emptyset P_n = \emptyset 0.80[0.85\mathrm{f'}_{\mathrm{c}}(\mathrm{A_g} - \mathrm{A_{st}}) + \mathrm{f_y A_{st}}]$$

$$(0.65)(329.52)\,k = (0.65)(0.80)[0.85(4\text{ ksi})(\mathrm{A_g} - 0.03\mathrm{A_g}) + (60\text{ ksi})0.03\mathrm{A_g}]$$

$$A_g = 80.79\,in^2$$

The area used by the SAP2000 model is 81 in^2 > 80.79 in^2. The design is safe as long as 30% reinforcing bars are used.

Using the Equations given by the interaction diagram in Figure 14:

$$K_n = \frac{P_n}{f'_c A_g}$$

$$P_n = \frac{P_u}{\emptyset} = \frac{329.52\,k}{0.65} = 507\,k$$

$$K_n = \frac{507\ k}{4\ ksi\ (81\ in^2)} = 1.56$$

$$R_n = \frac{P_n e}{f'_c A_g h}$$

$$M_n = \frac{M_u}{\emptyset} = \frac{2.17\ k-ft}{0.65} = 3.34\ \ k-ft$$

$$e = \frac{M_n}{P_n} = \frac{3.34\ k-ft}{507\ k}\left(\frac{12\ in}{1\ ft}\right) = 0.079\ in$$

$$R_n = \frac{507\ k\ (0.079\ in)}{(4\ ksi)(81\ in^2)(9\ in)} = 0.0137$$

$$\gamma = \frac{clear\ distance\ between\ confinement\ bars}{total\ length\ of\ column} = \frac{6\ in}{9\ in} = 0.667$$

After interpolating both graphs in Appendix B11 and B12, the value for the steel percentage ρ was around 0.0525

$$Reinforcement\ Area = \rho A_g = 0.0525\ (81\ in^2) = 4.25\ in^2$$

Final column design for axial and bending:

Use 10 #6 bars which have an area of 4.42 in^2 > 4.25 in^2

4.2 Results and Comparison of the Frame Analysis

The structure was re-analyzed with the exact same frame section dimensions used previously; the only difference is that the effect of the earthquake was taken into account using Equation 29. The results for each table in this section can be found in Appendices B13-B16

Table 4.1 – Bending Moment in Critical Beam Comparison

BEAM	Seismic Effect NOT Considered	Seismic Effect Considered
Positive Bending Moment	115.5 k-ft	225.2 k-ft
Negative Bending Moment	158.6 k-ft	321.12 k-ft

Table 4.2 – Shear Force in Critical Beam Comparison

BEAM	Seismic Effect NOT considered	Seismic Effect Considered
Shear Force	58.4 k-ft	79.1 k-ft

Table 4.3 – Beam Comparison

COLUMN	Seismic Effect NOT considered	Seismic Effect Considered
Axial Load	329.52 k	316.44 k
Bending Moment	2.17 k-ft	206.7 k-ft

Table 4.4 – Defection Check of the Centroid of Each Joint

	N-S Direction			E-W Directions			CHECK
Story	δ_{xe} (ft)	δ_x (ft)	Δ(in)	δ_{xe} (ft)	δ_x (ft)	Δ (in)	Δ_a (in)
Roof	0.2577	0.5154	0.648	0.0126	0.0252	0.0056	2.52
3	0.2307	0.4614	1.14	0.0098	0.0196	0.008	2.52
2	0.1832	0.3664	4.3968	0.0058	0.0116	0.0116	2.52
1	0	0	0	0	0	0	2.52

It can be seen that the second story deflects more than the allowable 2.52 inch

4.3 Discussion of the results

This shows how catastrophic it is to ignore the effect of an earthquake even in city that experiences a low frequency of earthquakes. If a seismic activity were to occur in the city of Oxford, the structural design of the building shown in section 4.1 would undergo an inevitable destruction due to the huge deflection and increased bending and axial stress.

The building was initially designed to carry just a gravity load, but when a combination of lateral loads simultaneously attacks the structure, it experiences a second combination of reactions. It is helpful to visualize this scenario:

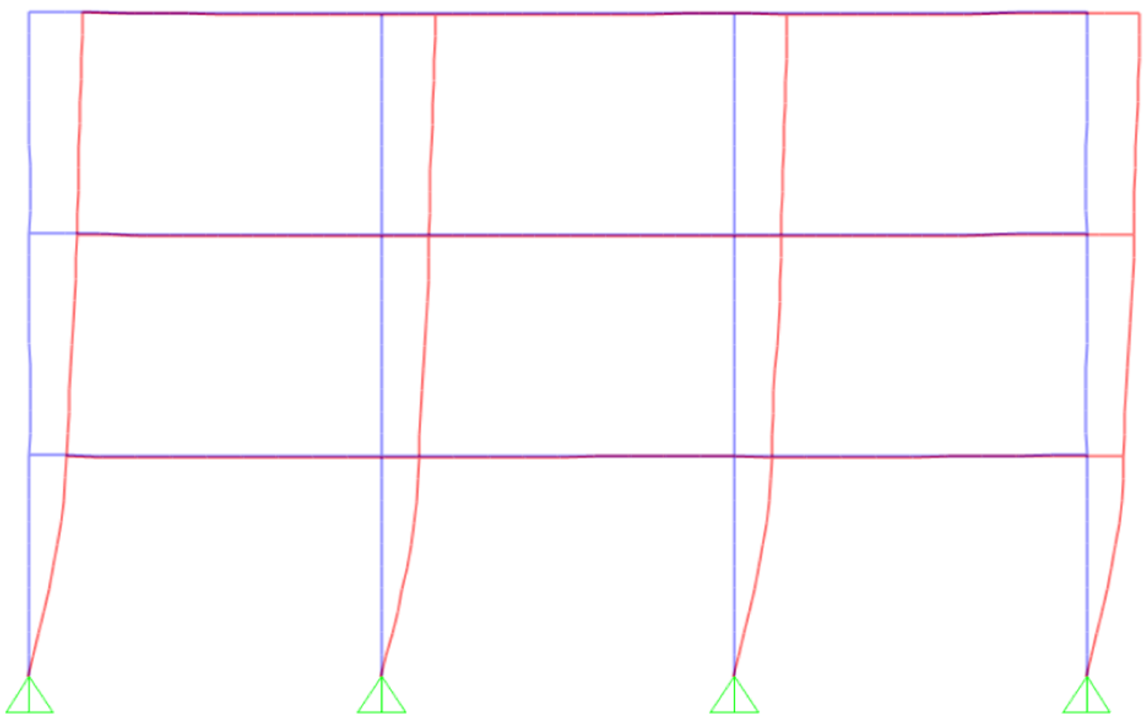

Figure [16] – Visualization of a Frame Undergoing Lateral Load and Gravity Loads at the Same Time

From Figure [16], several key differences can be seen when a frame experiences lateral load. The beams do not just bend, the ends also rotate along. The columns at the first floor experience a double curvature instead of a single curvature. This completely interferes with determining if the column is slender or not as stated in Equation 19.

From Table 4.1, the bending moment increases by more than a double when an earthquake hits the building. The building was initially designed to only withstand a 237.6 k-ft moment.

Initially, every beam was designed to resist a shear force of 51.4 kips according to the critical beam in the top right of the frame (Appendix B4). With the lateral load, the critical beam becomes the member at the bottom right of the frame, and experiences 69.4 kips of shear as seen in Table 4.2. That is 18,000 more pounds of shear force that has not been accounted for at all during the design with no seismic effect consideration. Therefore, the design should be rejected and the calculation for the stirrup design should be re-iterated with a shear force of 69.4 kips to ensure adequate stirrups are placed to carry the 18 extra kips.

The column experiences about 55 more kips of axial load as seen in Table 4.3. This does not make much of a difference when the gross area A_g is re-calculated because the load is still relatively small regardless. The enormous difference comes from the bending moment, which goes from 6.61 k-ft to 426.6 k-ft. This would change the value of R_n tremendously and require much bigger reinforcing bars.

The columns were initially defined to be 9 inches by 9 inches. This was enough to resist gravity loads, but when lateral loads were applied-the deflections as seen in Table

4.4 were tremendous. This is not surprising, as the columns were the smallest possible dimension, meaning the smallest possible moment of inertia, making the column most prone to deflection.

SUMMARY AND CONCLUSIONS

It can be observed from Figure [16] that both the beams and columns are bending at a larger magnitude, which directly relates to why the seismic consideration induces much larger values for the bending moment in both the columns and the beams. The objective of this study has been successfully accomplished. The effect of an earthquake is extremely substantial that it cannot simply be ignored for the senior capstone project. This results in Tables 4.1-4.3 server as evidence for the reason behind the International Building Code classifying Oxford as a Risk Category III. Consequently, it can be seen why the International Residential Code regards Oxford as seismic design category C in Figure 17

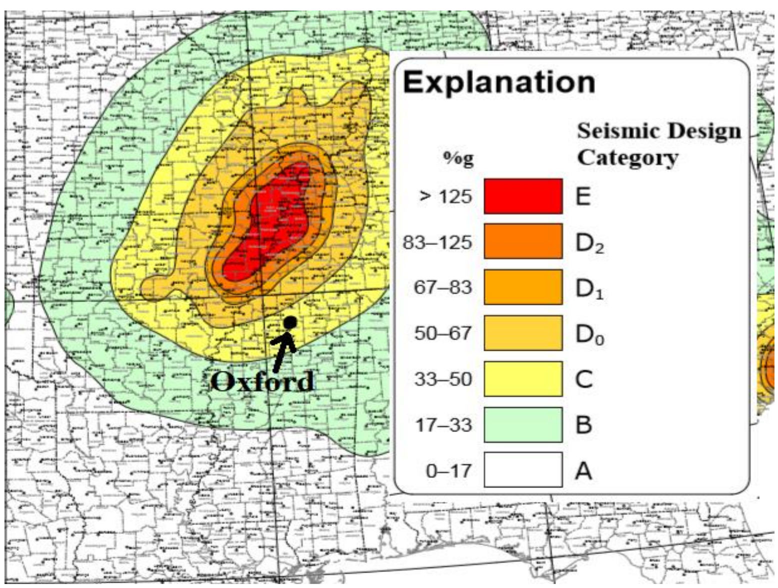

Figure [17] – Seismic Design Category map of the region around northern Mississippi

When designing a building, regardless of the location, the seismic activity should always be considered as it can have a huge impact on the design. The frequency of earthquake occurrence does not necessarily guarantee whether or not an earthquake will take place. Therefore, it is highly beneficial for every civil engineering student to learn the step-by-step process on how to evaluate the lateral load based on the location and be able to design for it using the code provisions of ACI, ASCE and IBC. Structural engineers always account for the seismic effect in the real world, therefore the students who are interested in the structural engineering field should be familiar with seismic design as well.

RECOMMENDATIONS

The technique used to determine the effect of the seismic loads was an overdesign. This is because the selected frame from Figure [4] was structurally analyzed without considering the stiffness of the slab placed on top of each beam. Had the slab stiffness been taken into account, the amount of lateral restrained on the frame would decrease by a large amount. This decreased amount of lateral movement would lower the amount of bending and thus lower the bending moment values.

All work was based on a 2-Dimensional analysis. If a complete 3-Dimensional analysis of the NOLA was done, the moment values would be less since the out-of-plane beams could have contributed to resisting the lateral force on the joints, which would lower the amount of lateral movement and thus the amount of bending moment.

A complete 3-Dimensional model as well as a slab stiffness consideration is recommended for future work regarding the topic of this thesis.

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[13] ASCE 7-16, 12.8.2.1 Approximate Fundamental Period, Equation 12.8-7, American Society of Civil Engineers, 2017.

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[25] ASCE 7-16, 12.8.5 Overturning, Equation 12.8-15

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[28] ASCE 7-16, 2.3 LOAD COMBINATIONS FOR STRENGTH DESIGN, 2.3.1 Basic Combinations, American Society of Civil Engineers, 2017.

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APPENDICES

Appendix A – Excel Sheet Used to Derive the Seismic Design Shear for each Story

Equivalent Lateral Force Procedures					
Calculation of seismic response coefficient & Base Shear					
R	**=**	3	*(Ordinary Reinforced Concrete Moment Frame)*		*(ASCE 7-16, Table 12.2-1)*
Ie	**=**	1.25	(seismic)	(importance factor)	
C_s	=	$S_{ds}/(R/I)$	0.15		*(ASCE 7-16, 12.8.1.1)*
					From Preliminary Design
$\mathbf{C_d}$		2.5	*(Ordinary Reinforced Concrete Moment Frame)*		

Appendix A1 – Part 1 of the Excel Calculation for Deriving Seismic Design Shear

Determination of Fundamental Period					*(ASCE 7-16, 12.8.2)*
Ta = $C_t h_n^x$		0.64			
		C_t	0.016	RC Frame	*(ASCE 7-16, Table 12.8-2)*
		x	0.9	RC Frame	
Ta =	0.64				
This Ta should not exceed (<u>**Cu**</u>)					*(ASCE 7-16, 12.8.2)*
		Cu	1.6	For S_{D1}>0.15	*(ASCE 7-16, Table 12.8-1)*
			OK		

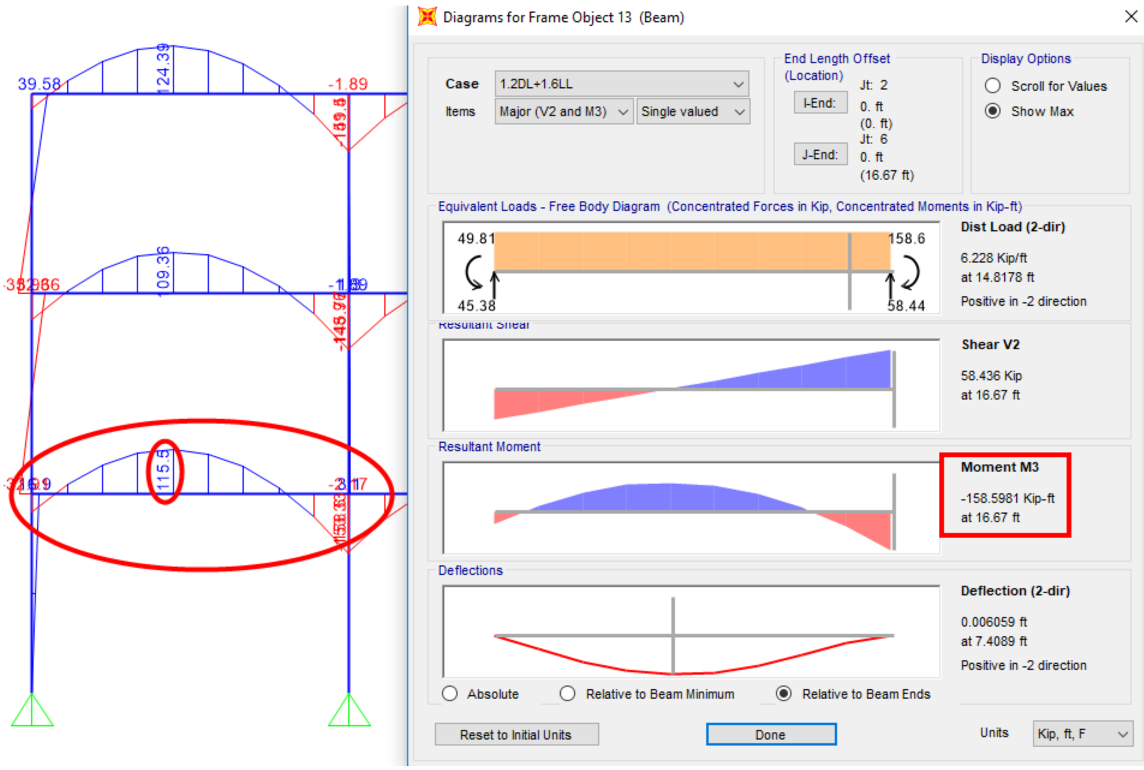
Appendix A2 – Part 2 of the Excel Calculation for Deriving Seismic Design Shear

Find Base Shear V					
<u>Weight per floor must be found first</u>					
Concrete weight assuming plain concrete without rebar					
W_{slab}	796.875	kips	$W_{ground\ floor}$	31.04592	*(8.5" slab)*
W_{beam}	306.3113	kips	$W_{first\ floor}$	1158.966	*(Using beam design of 14"x24.5")*
$W_{columns}$	30.625		$W_{second\ floor}$	1158.966	*(Assume all 10"x10" columns)*
W_{story}	1133.8113	kips	W_{roof}	1127.92	
W	3476.897113	kips	(Seismic Weight)		
V	522.3072108				
<u>Steel rebar weights</u>					
$W_{column\ rebars}$	0.420924328	kips	$W_{slab\ rebars}$	18.2693	kips
$W_{beam\ rebars}$	6.46419504	kips			
Distribute V over the height of Structure					*(ASCE 7-16, Table 12.8.11- Table 12.8.12)*

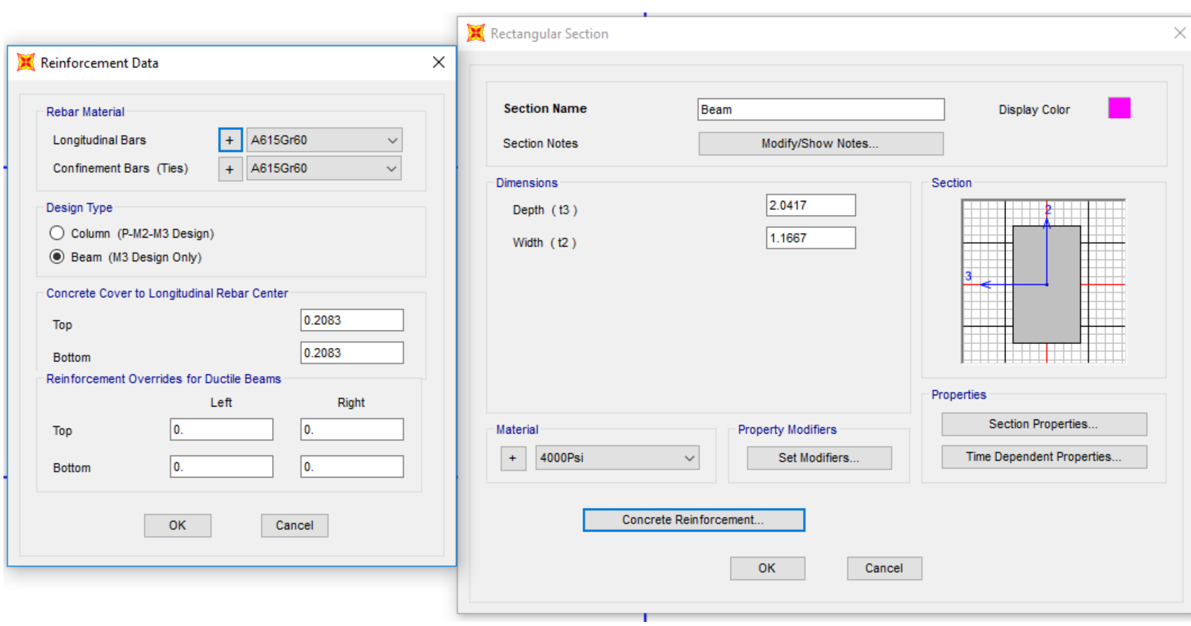
Story	Wx (kips)	hx (ft)	Wx*hx^k	Cvx	Fx (kips)	Vx (kips)
1		0.0			0	522
2	522	10.5	5484	0.17	87	435
3	522	21.0	10968	0.33	174	261
Roof	522	31.5	16453	0.50	261	0
	1567		32905		522	

Appendix A3 – Part 2 of the Excel Calculation for Deriving Seismic Design Shear

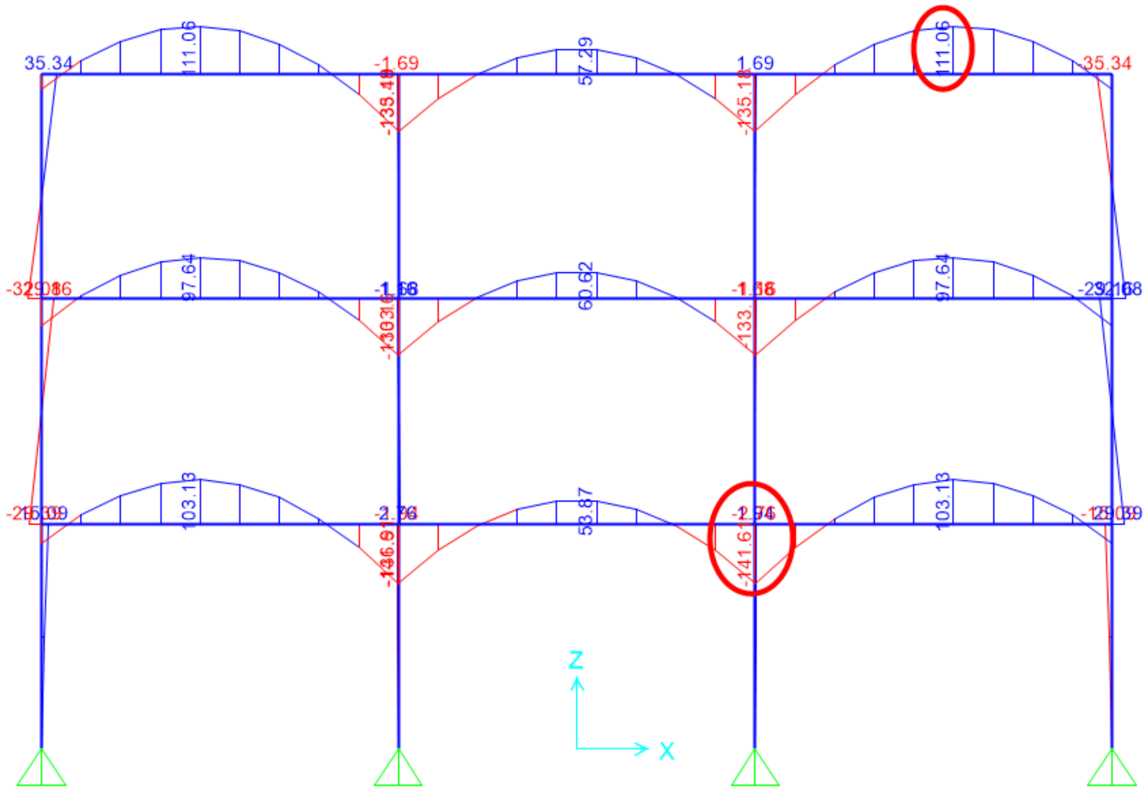
Appendix B – SAP2000 Results and Interaction Diagram Interpolation



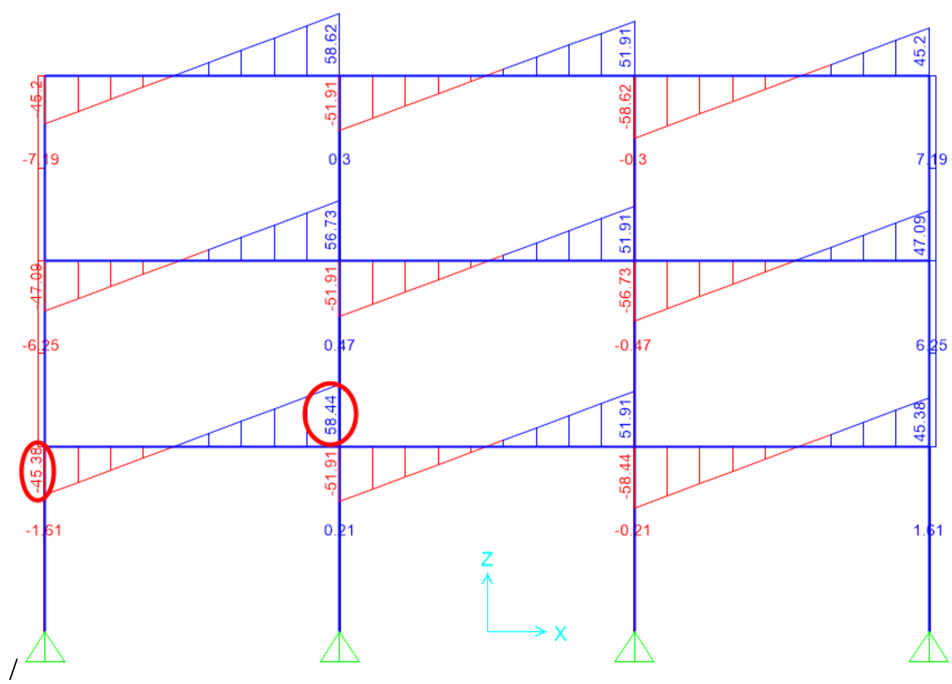
Appendix B1 – Critical Bending Moment Results of Dead and Live Load Case Using the Initial Beam Dimension Assumption



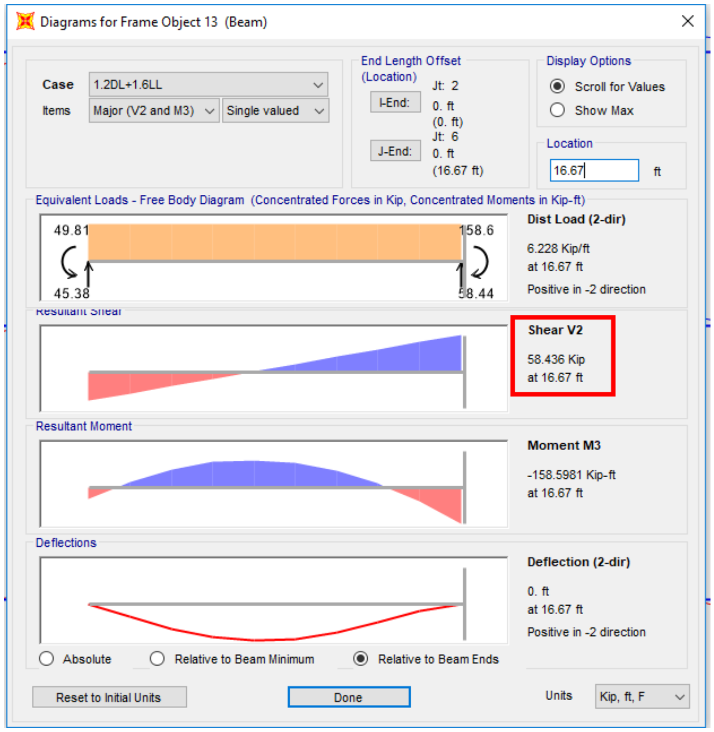
Appendix B2 – Final Frame definition of Beam



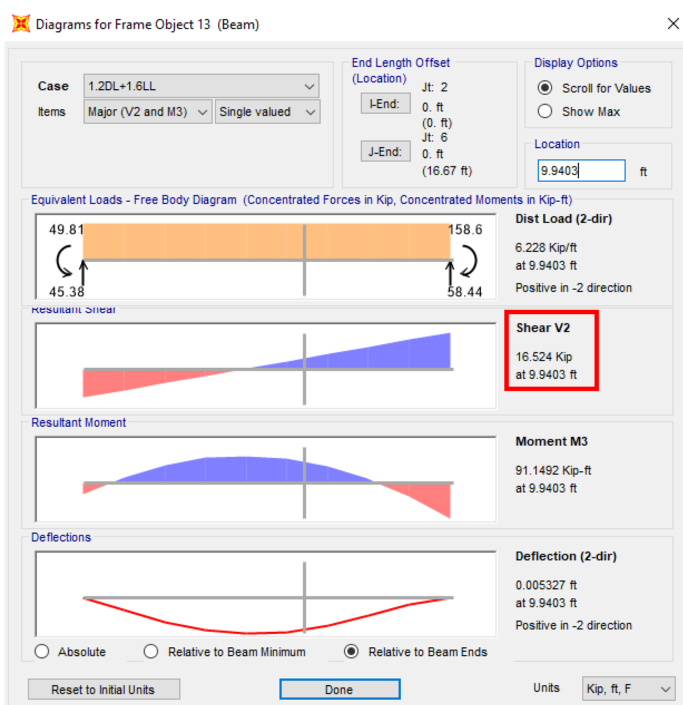
Appendix B3 – Final Critical Bending Moment Results for the Most Vulnerable Beams Using 24.5” Depth and 14” Base Width Beam



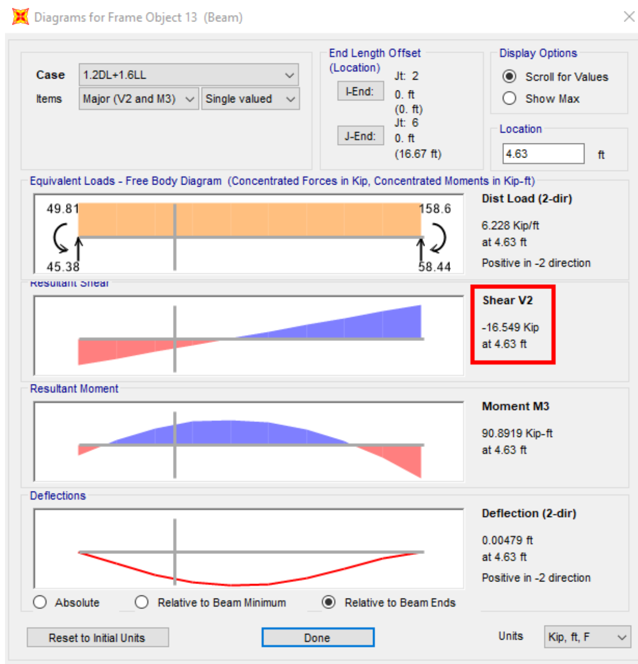
Appendix B4 - Final Critical Shear Result for the Most Vulnerable Beam Using 24.5” Depth and 14” Base Width Beam



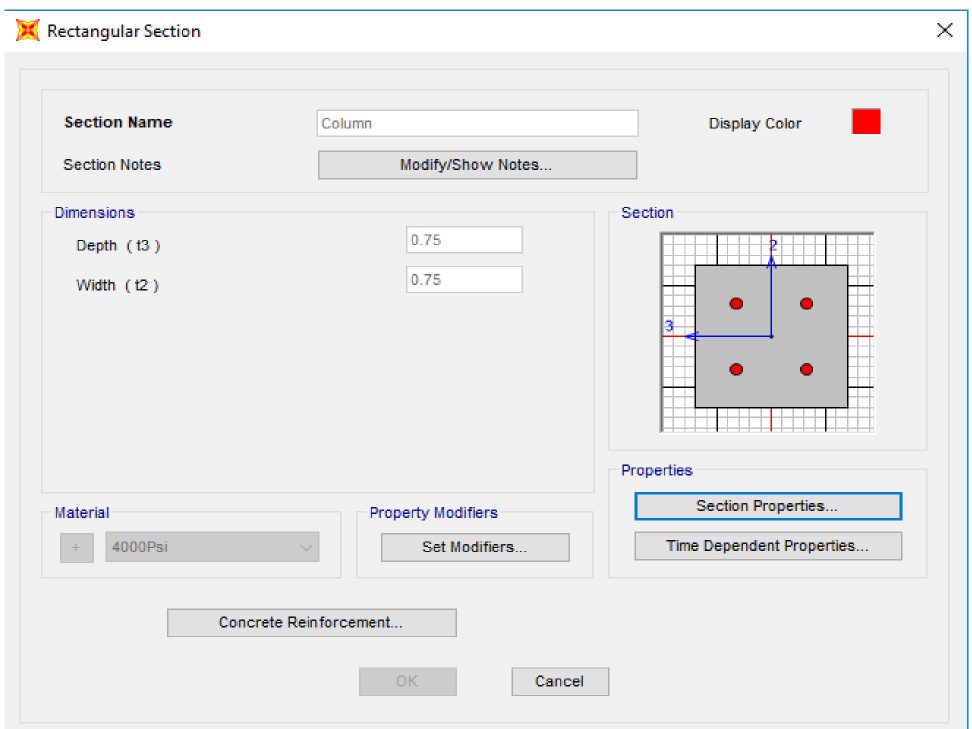
Appendix B5 – Ultimate Shear at a Distance equal to d from the Face of the Support



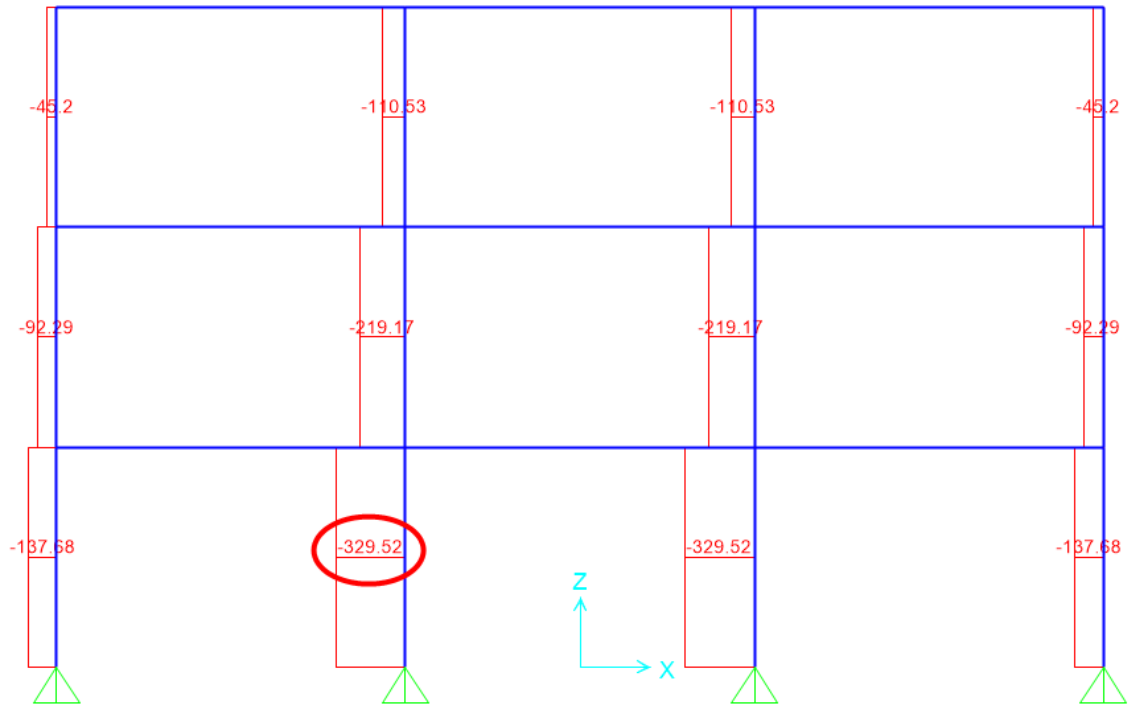
Appendix B6 – Stirrup Termination Location at Approximately 8.6 ft Away from the Left Support (Where The Shear Experienced by the Beam = $\frac{1}{2}\emptyset V_c$ = 16.5 k)



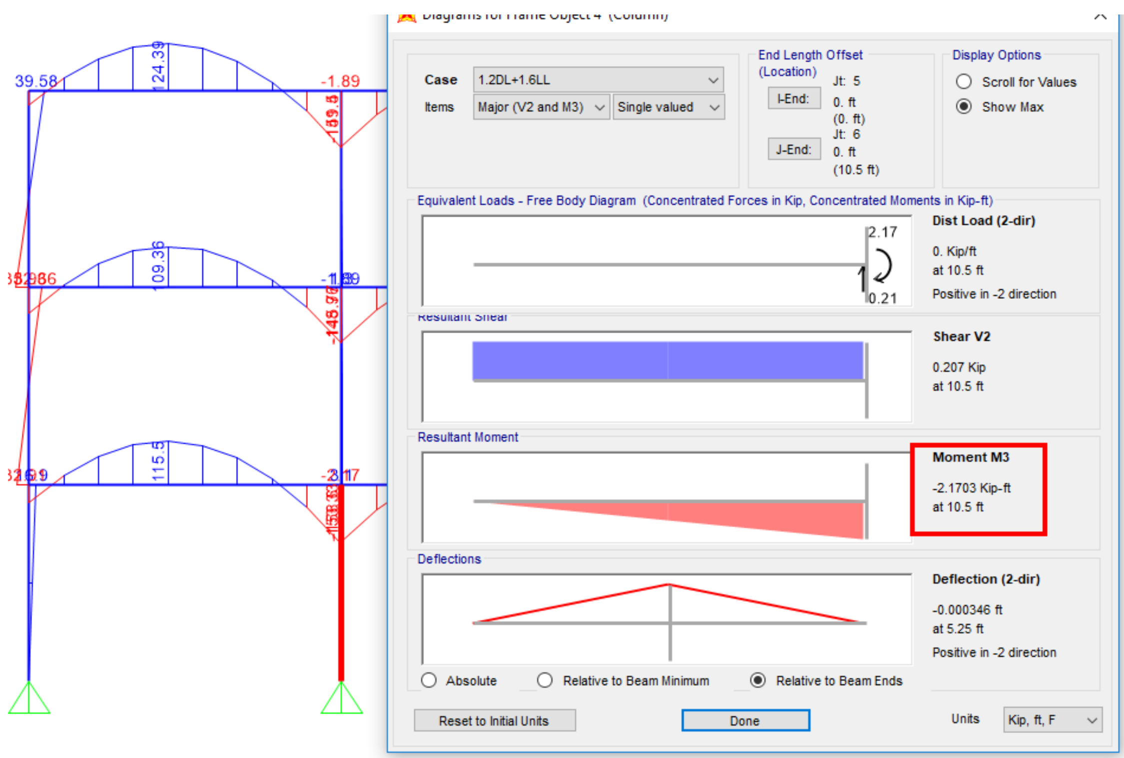
Appendix B7 – Stirrup Re-Applied at Approximately 21.14 ft Away from the Left Support (Where the Shear Experienced by the Beam = $\frac{1}{2}\emptyset V_c$ = 16.5 k)



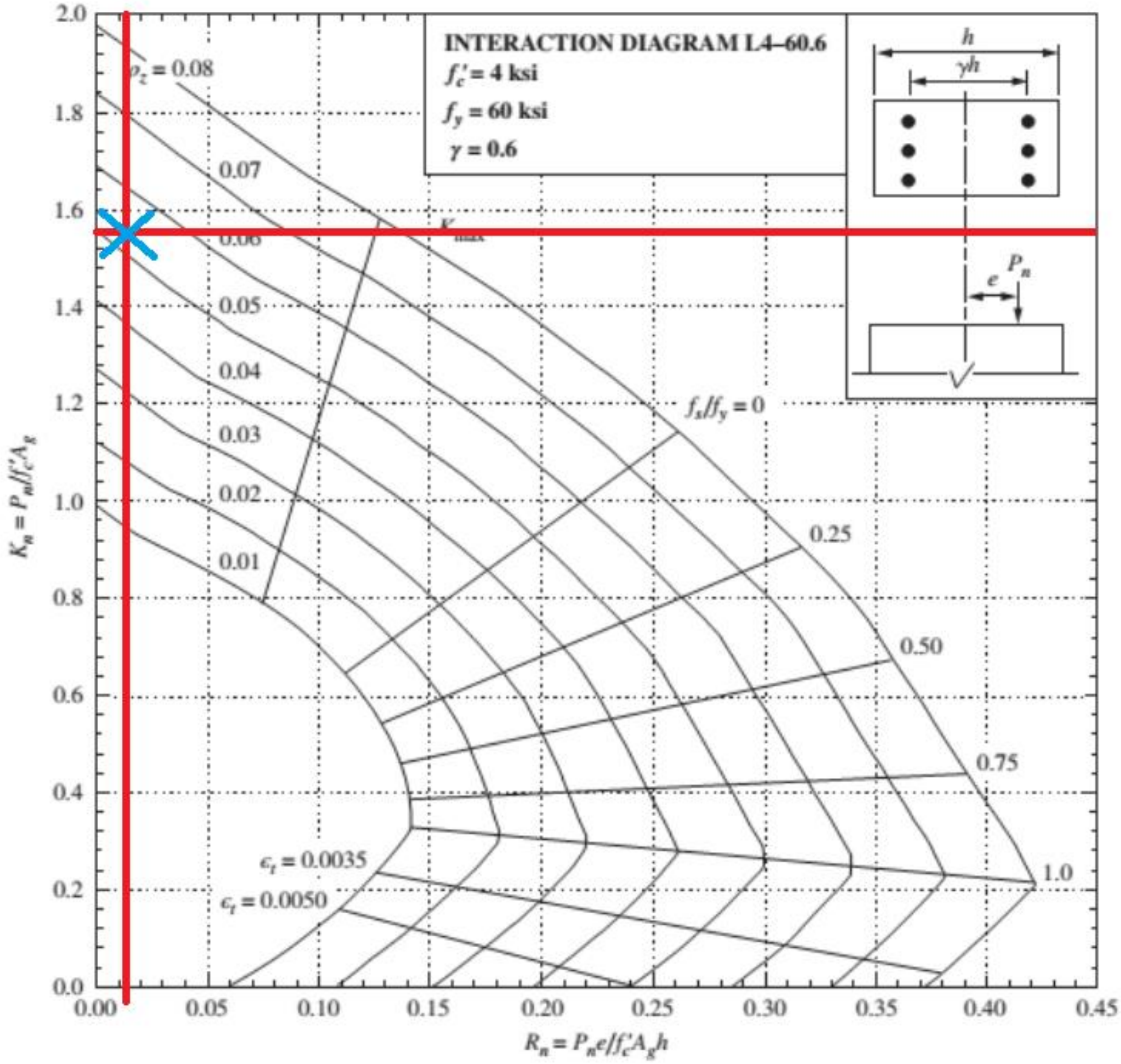
Appendix B8 – Final Frame Definition of a Column



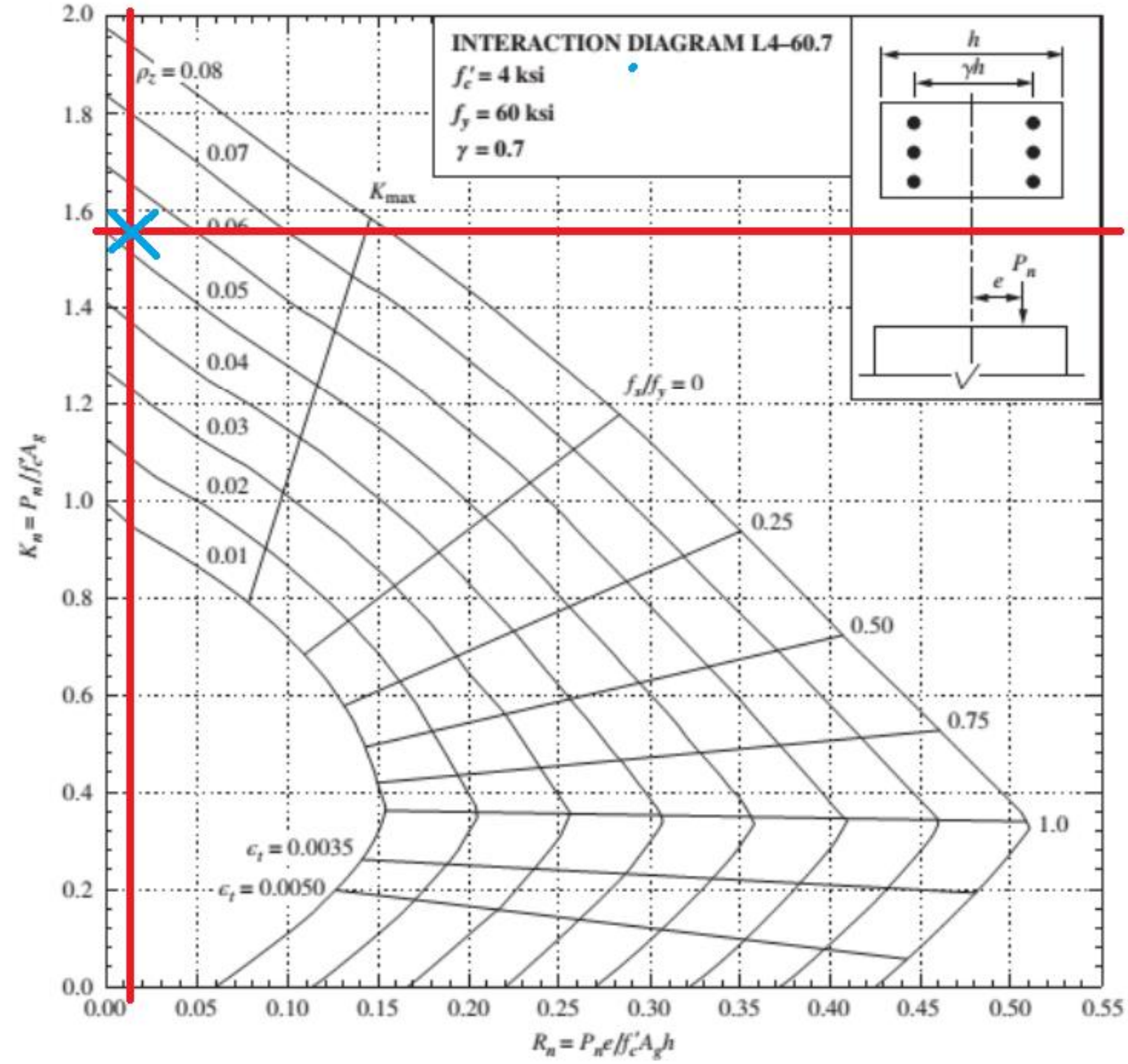
Appendix B9 - Final Critical Axial Load Result for the Most Vulnerable Column Using 24.5” Depth and 14” Base Width Beam



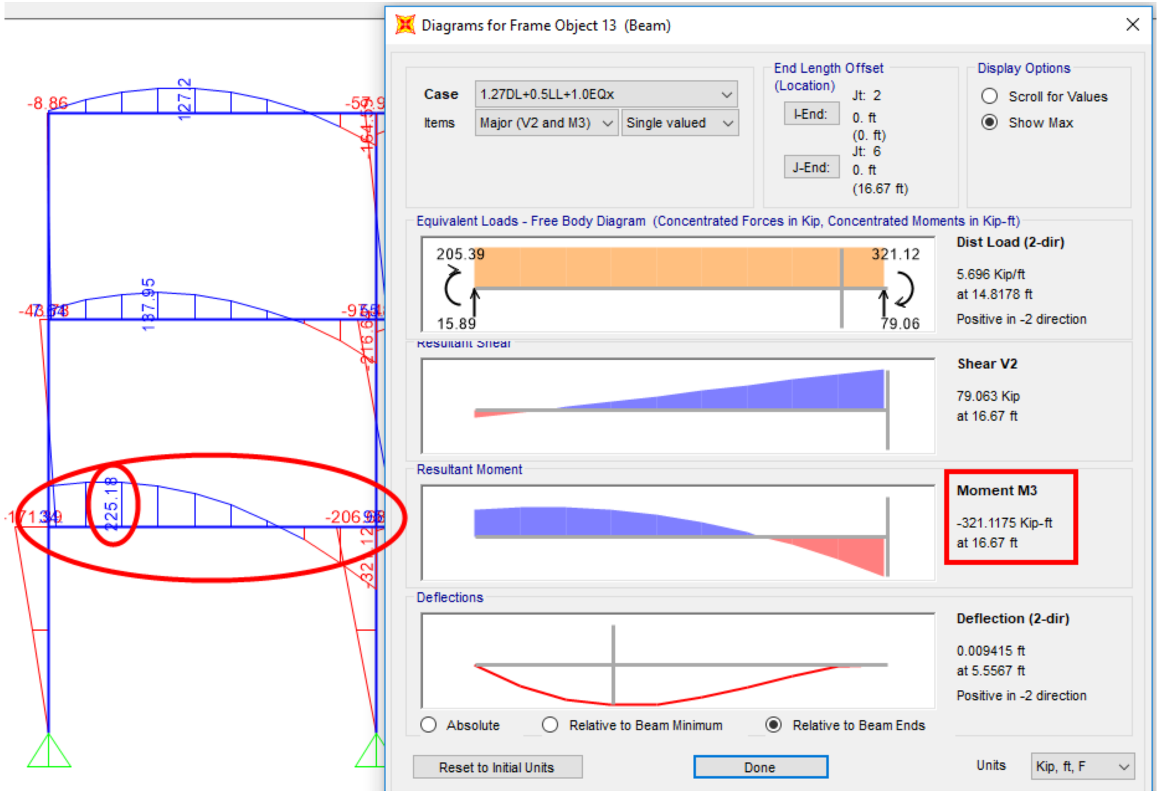
Appendix B10 - Final Critical Bending Moment Result for the Most Vulnerable Columns Using 24.5” Depth and 14” Base width Beam



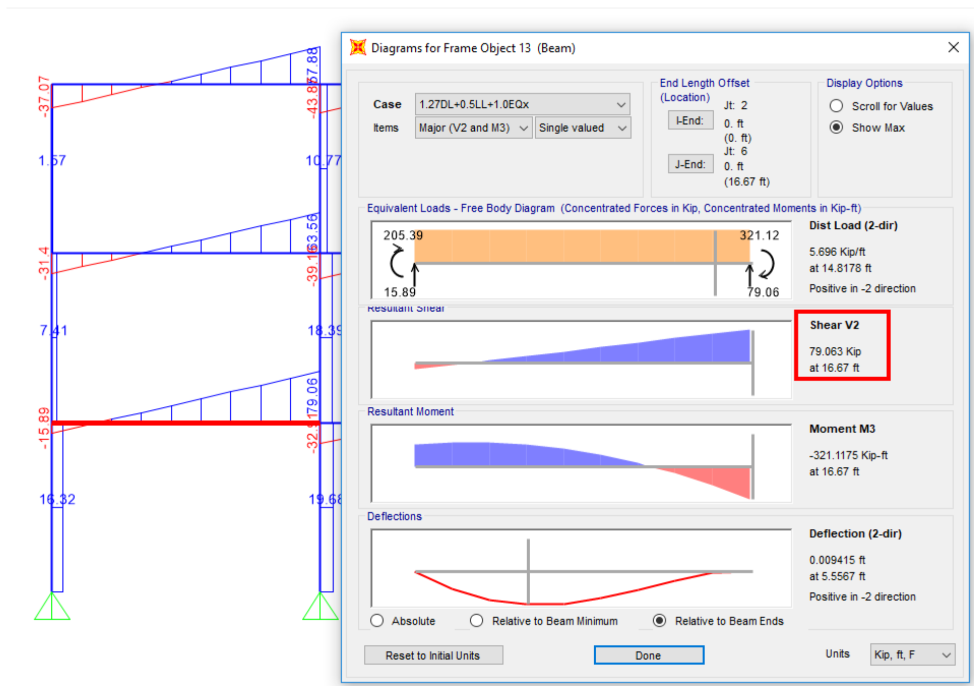
Appendix B11 – Interpolating the Steel Percentage ρ from the Interaction Diagram for $\gamma = 0.6$ [30]



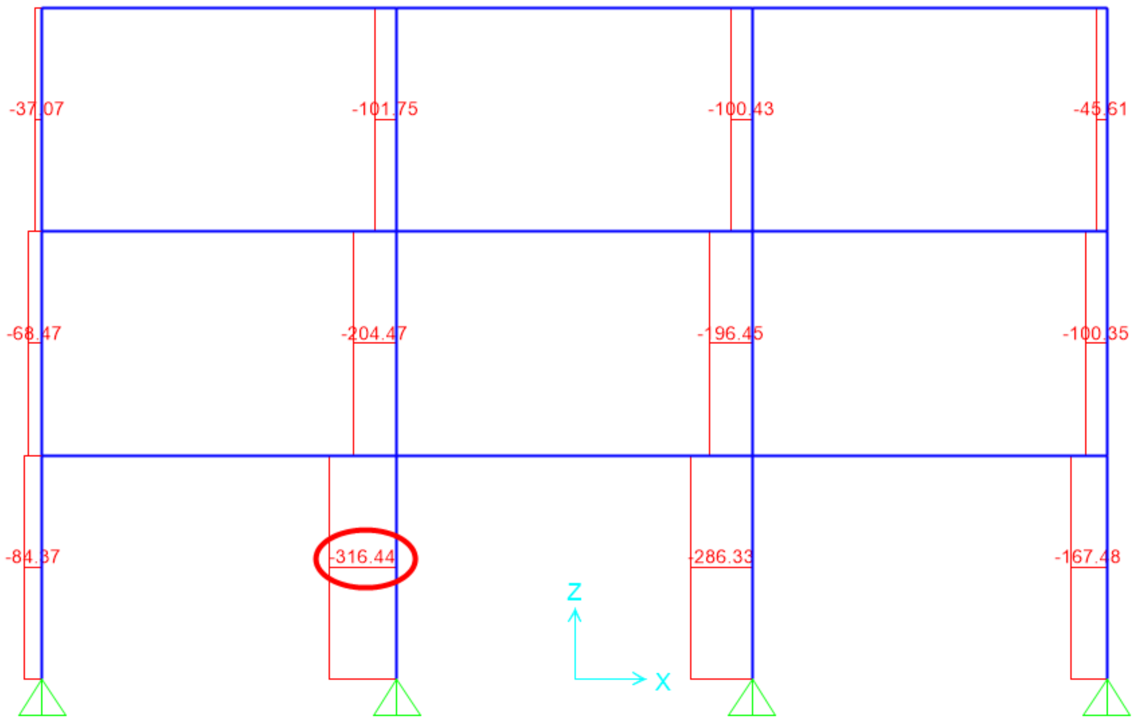
Appendix B12 – Interpolating the Steel Percentage ρ from the Interaction Diagram for γ = 0.7 [31]



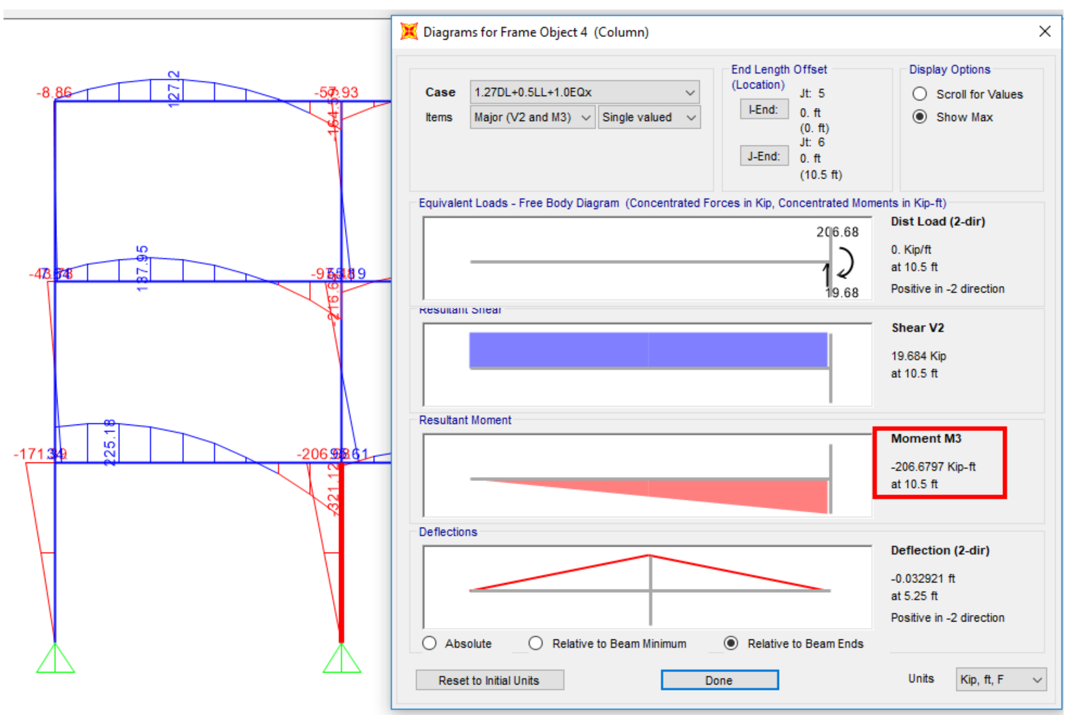
Appendix B13 - Final Critical Bending Moment Result for the Most Affected Beam After Lateral Loading



Appendix B14 - Final Critical Shear Result for the Most Affected Beam After Lateral Loading



Appendix B15 - Final Critical Axial Load Result for the Most Affected Column After Lateral Loading



Appendix B16 - Final Critical Bending Moment Result for the Most Affected Column After Lateral Loading